



BURGESS & NIPLE

Structural Investigation

Lazarus Furniture Warehouse Whittier Peninsula Redevelopment

City of Columbus, Ohio

November 1, 2005

Since 1912



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INTRODUCTION

This report summarizes the work that Burgess & Niple, Inc. (B&N) conducted in analyzing those portions of the former Lazarus furniture Warehouse that are to remain as part of the redevelopment of the Whittier Peninsula area. The purpose of the analysis was to evaluate whether the structure meets current building and design code standards, and if not, to make recommendations for retrofitting the structure to bring it up to those standards. Information concerning the structure was collected by field measurements, and the analysis to determine the structure's design loads used computer modeling and hand calculations.

On August 26 and 29, 2005, a two-member team from B&N conducted field measurements of the eastern third of the warehouse. Measurements of the bays were taken to establish building size. The location, orientation, and height of columns were recorded. The dimensions of framing members, including beams, joists, columns, and bracing, were measured. A two-member crew from B&N's Bridge Inspection Section was brought in on September 8, 2005 to measure the truss depth and dimensions of truss components. Type and quantity of typical framing connections and wall anchorages were noted. Second floor slab thickness and metal roof deck dimensions were recorded. The information gathered was used to model the building so a structural analysis could be conducted.

Included in this report are conceptual recommendations on how to upgrade the existing structural system to comply with the current building and design code requirements. Preliminary retrofit details, a preliminary cost estimate, and sample modeling data used for evaluation of the structure are also included in the body of this report. Assumptions regarding loading and building components are stated. Not included in this report are any conclusions on the suitability of the foundations, since the invasive field work necessary to ascertain this information was beyond the scope of this study.

BUILDING OVERVIEW

The building is a two-story steel-framed structure built in 1955, enclosed by masonry walls. It has overall dimensions of approximately 318 feet by 308 feet, which include a one-story “high bay” area 291 feet by 69 feet. The high bay area is framed with trusses that create a column free interior. The two-story portion of the building generally is composed of 20'-0"-wide bays in the east/west direction and 22'-2" bays in the north/south direction. The second floor slab is 4 inches thick, and the roof deck is 1 inch deep. The roof has a slope of approximately ½” per foot.

A column grid was created so areas in plan could be conveniently located as well as provide the layout for beams, columns, and open bay areas (Figures 1, 2, and 3 in Appendix A show schematic framing layouts at the second floor, lower roof, and upper roof, respectively). Column line 1 defines the northern face of the building, while column line 13 sets the southern face of the building. Column line 12 is where the roof steps down to a one story structure and remains so all the way to column line 13.

STRUCTURAL ANALYSIS

The building was analyzed using the software packages RAMsteel™ and STAADPro™, along with calculations done by hand. A copy of the STAADPro™ input is found in Appendix B. The 2005 edition of the *Ohio Building Code* and the 2002 edition of *ASCE 7 – Minimum Design Loads for Buildings and Other Structures* were used to determine loading and structural performance requirements.

LOAD CRITERIA AND DESIGN ASSUMPTIONS

Dead load

Second Floor: 60 psf (includes 10 psf collateral load)

Roof: 22.5 psf (includes 10 psf collateral load)

Live Load

Floor Live load: 60 psf (40 psf residential plus 20 psf partition load)

Roof Live load: 20 psf

Snow load: 20 psf, plus a 4.75 foot deep drift on the lower roof 19 feet wide with a maximum intensity of 78.9 psf.

Wind Load

Wind speed, V: 90 mph

Exposure: B

Importance Factor I_w : 1.0

Maximum load on wall: 12.8 psf

Maximum uplift on roof: -15.4 psf

Components & Cladding

	Zone	Area, ft ²	Pressure, psf	Suction, psf
Roof	1	10	5.9	-14.6
	1	100	4.7	-13.3
	2	10	5.9	-24.4
	2	100	4.7	-15.8
	3	10	5.9	-36.8
	3	100	4.7	-15.8
Wall	4	10	14.6	-15.8
	4	100	12.4	-13.6
	5	10	14.6	-19.5
	5	100	10.9	-12.1

Seismic Load

Seismic Use Group: I

Seismic Design Category: B

Site Class: D

Importance Factor, I_E : 1.0

$S_{DS} = 0.178$

$S_{D1} = 0.106$

Lateral load resisting System: Ordinary reinforced concrete shear walls and ordinary concentrically braced steel frame (South wall only).

$R = 5$

Seismic Base Shear: 349 kips

Deflection Criteria

Deflection due to live load: L/360 for floor, L/240 for roof (assumes nonplaster ceiling)

Deflection due to total load: L/240 for floor, L/180 for roof (assumes nonplaster ceiling)

Assumed Material Properties

Steel (existing)

Yield Strength: 36 ksi
Tensile Strength: 58 ksi
Concrete (proposed)
 $f'_c = 4$ ksi
Masonry (existing)
 $f'_m = 1.5$ ksi

Notes on Loading Assumptions:

With the exception of the second floor live load, which anticipates the building's use as a residential facility, and collateral loads which posit the inclusion of a ceiling, HVAC components, a sprinkler system and other fixtures typically found in an occupied space, the loads on the structure were determined based on the present state of the building. As such, they do not take into account possible modifications to the roof (i.e., increase of live load, different roofing material, changes in insulation thickness, etc.) or the effects of possible additions to, or replacement of, wall cladding, openings, or other substantial architectural features that might occur when the renovation plans are finalized.

SUMMARY OF FINDINGS

Examination of the Building Subjected to Gravity (Vertical) Loads

Roof Framing

The roof framing is at two elevations, referred to here as the main roof and lower roof, which occupies the southern bay of the building. The main roof framing requires no upgrades. It should be noted, however, that the beams along column lines A and S do not have the capability to support any additional load. If roof modifications increase weight or change of use increases the roof live load, then those beams will have to be reinforced or replaced.

The joists of the lower roof nearest to the change in elevation fail when subjected to the code required snow drift. It is apparent that the framing did not account for snow drift in the original design. One joist in each twenty foot wide bay fails in bending. In addition, two vertical members of the trusses in the southern bay, one at each end, are failing in compression. See Figure 4 in Appendix A for location.

Second Floor Framing

Joists and beams do not require reinforcement. Note that this assumes that bays currently open to the first floor below remain open and will not be filled in with structural framing and a concrete slab.

Columns

Columns do not require reinforcement.

Examination of the Building Subjected to Lateral Loads

The building resists lateral loads (i.e., wind and seismic forces) in the east/west direction by means of steel braced frames, located in four bays on the north and south walls (bays C-D, G-H, L-M, and Q-R). Horizontal bracing at the roof level distributes lateral load to each bay with vertical bracing at the ends.

The horizontal bracing is not fully effective because the change in elevation from the main to lower roof introduces a vertical discontinuity in the lateral load resisting system. This gap in the system needs to be bridged, or the columns will fail in weak-axis bending.

The X-bracing in the bays of the south wall is undersized (i.e., it violates minimum slenderness requirements).

The bracing in the north bays is only partial height; that is to say, it does not extend full height or down to the foundation system. For that matter, in three of the four braced bays it does not even extend down to the second floor because it is situated in bays where the track of an overhead door would interfere with the framing. The bracing system on the north side is inadequate for the current loads.

It is possible that lateral loads are being transferred into the masonry at the second floor elevation and that the exterior wall was intended to be used as a lateral load resisting element. In this scenario, the load path would take the load from the bracing into the columns, to the second floor beam, to the strap anchors that tie the masonry to the framing, to the masonry wall, and down to the foundation. Assuming that the anchors, which have an unverifiable shear capacity, are adequate, then the wall would have to be able to function as a shear wall. However, the wall in its current state cannot properly fulfill this function. The exterior walls are almost certainly unreinforced, which was still the common practice during the period when the building was constructed. For example, provisions for designing reinforced masonry were not published until 1960 (*Building Code Requirements for Reinforced Masonry*), and a 1951 edition of *Audel's Masons and Builders Guide* does not mention reinforcing masonry whatsoever. While the building code does permit unreinforced masonry to be used as the lateral force resisting system in limited circumstances, the design is penalized. For instance, the design seismic load that must be resisted by unreinforced masonry shear walls is twice that of the load for properly reinforced and detailed masonry walls, and more than three times the magnitude that reinforced concrete shear walls must resist. Simply put, there is not enough wall for the building to rely upon this type of lateral load resisting system.

For loads applied in the north/south direction, the lateral load resisting system consists of steel moment frames at each column line. Because a frame was included at each column line,

horizontal bracing was not necessary to distribute loads at the roof. Semi-rigid, or “wind” connections at the roof level along column lines 1 and 12 produce a frame that includes the beam and column on either side of the connection. There are no such connections at the second floor level.

The semi-rigid moment connections of the building frame are not capable of handling the induced forces from current lateral loads. The following excerpt from *Structural Renovation of Buildings*, a resource for engineers renovating existing buildings, cogently illustrates the inherent problems of this type of system:

Another solution for assuring stability in the older iron and structural steel building involved partially restrained (semi-rigid) connections. These “wind” connections, as they were called, were supposed to be rigid enough to resist lateral loads, but flexible enough to allow the beams to behave as simply supported members under gravity loads. ...The simplicity and low cost of partially restrained connections is counterbalanced by their relatively poor energy dissipation, because the joint is weaker than the connected members and plastic hinges cannot be formed in the beam, as is desirable. Also, buildings that use these connections tend to undergo large story drifts. . .

Charles W. Roeder, in a study of the seismic performance of older steel frames with semi-rigid connections, concluded that, theoretically, these frames could not provide the level of seismic resistance required by the modern building codes. How is this conclusion reconciled with the relatively good real-life experience with partially restrained connections? Roeder’s explanation is simple: none of the buildings he studied had actually to withstand an extreme earthquake in its lifetime. Also, in all those buildings, the framing was...surrounded by unreinforced masonry walls – elements that were much more rigid than the steel frame.

- Alexander Newman, *Structural Renovation of Buildings*, 2001, p. 632.

As it happens, even if the moment connections themselves are adequate, other constituents of the frame are not. In the absence of information to the contrary, frames are assumed to have columns that are pinned at the base (permitting rotation). This contributes to the overall flexibility of the system and increases story drift. And as indicated above, the story drift resulting from the use of these frames is excessive, in this case the story drift is 6.25", while the allowable story drift is only 3.12". In addition, the columns along column line 12 and 13 fail, along with the roof level beam that spans from line 11 to 12.

It might be possible that the base of the columns in the moment frame could legitimately be considered fixed at the base (a rigid support that permits no rotation), although since the

concrete floor slab covers the base of the column, this is not verifiable. But it is reasonable to surmise that columns that already participate in frame action are restrained at the base. If this is the case, the frame is still insufficient: the story drift becomes 4.61", and the columns along line 1 fail.

The building requires an alternative system for resisting lateral loads in the north/south direction.

Walls

The masonry walls are generally fastened to the building frame with strap anchors embedded in horizontal mortar joints along the height of each column vertically and at intervals of 3 to 4 feet at the second floor level horizontally. An exception to this is along the west side (column line A). This absence is evidently due to the west wall actually belonging to the adjacent building, and therefore being constructed prior to the building being considered. In addition, at the roof level of the east and west walls the masonry does not appear to be tied back to the framing. Anchorage of the wall is required at these locations and at other isolated locations where the anchor attached to a column has failed and pulled out of the masonry.

RECOMMENDATIONS FOR STRUCTURAL UPGRADE

1. At the lower roof level, an additional joist should be installed in each bay adjacent to the joist failing due to snow drift. This will relieve the existing joists of some of their respective load. Since this new joist will fall between truss panel points, the top chord of the truss must be reinforced (Figures 5 and 5a in Appendix A) to support the new concentrated load from the joist.
2. The failing vertical members of the truss must be reinforced to properly resist compression. Truss vertical reinforcement consists of increasing the section properties of the member by welding additional angles to the existing double angle member (Figure 6 in Appendix A).
3. Bracing in the vertical plane between the upper and lower roof framing along column line 12 should be installed in four bays (Figure 7 in Appendix A).
4. X-bracing in bays along the south side of the building should be replaced with more substantial members (Figure 8 in Appendix A).
5. The installation of shear walls on the north, east and west sides of the building is recommended. These will become the primary lateral load resisting system. Each of these three exterior walls requires three bays of the existing masonry to be reinforced to suitably perform when subjected to lateral loads from current versions of the *Ohio Building Code* and *ASCE 7 – Minimum Design Loads for Buildings and Other Structures*. Future collaboration with the architect is required to select a prospective location, preferably a bay with no openings in the wall. The south side of the building has existing bracing as its lateral load resisting component, and once upgraded, will work in concert with the shear walls.

It is recommended that walls designated as shear walls be reinforced with shotcrete (i.e., gunite). This will be less labor intensive and cheaper than reinforcing the masonry itself by installing vertical reinforcing bars. A typical application includes a continuous 3 or 4 inch thick layer of shotcrete sprayed on the face of the CMU. This is reinforced vertically and horizontally with a centrally located layer of rebar, for example bars of

3/8 inch to 1/2 inch in diameter. To ensure bonding of the concrete to the existing masonry, vertical ribs conceptually akin to hidden pilasters are inserted into the masonry at intervals of 6 to 8 feet (Figure 9 in Appendix A). These vertical ribs are reinforced similarly to concrete columns, with vertical bars and closed ties.

Application of the shotcrete should occur at the exterior face of the wall to facilitate construction, if possible. The east and west walls are exposed concrete masonry, and since they lack a brick veneer that the architect may wish to preserve, the exterior application should be feasible. Assuming that the brick on the northern exterior wall is to remain, the shotcrete application would have to be done on the interior face of the wall (Figure 10 in Appendix A). The figures and cost estimate reflect this assumption.

In order to transfer lateral forces into the shear walls, shear transfer connections are required at the roof and floor level. On the north side, the bottom chord of the “partial height” bracing is where the transfer occurs, since the shotcrete will only come up to the sill of the sash windows. But since the partial bracing is located in bays with overhead doors below, the shear should be positioned in a different bay. Drag struts are now required to transfer the shear from the bracing to the shear wall (Figure 11 in Appendix A). Figures 12 and 13 in Appendix A show typical shear transfer connections.

The forces in the shotcrete must be transferred to the foundation. Existing foundation configuration and condition is unknown at this time, but a 2'-0" wide wall footing and 4'-0" square column footings were assumed along the building perimeter. Cast in place concrete with adhesive set dowels into the existing foundation serve as a base for the shotcrete layer (Figures 14 and 15 in Appendix A). Note that excavation for the interior shotcrete application would necessitate removal of a portion of the existing slab on grade for the length of bay that is reinforced with concrete. Again, an exterior shotcrete application would be more cost effective.

6. At the roof level of the east and west walls, the masonry should be tied back to the framing per Figure 16 in Appendix A, and back to columns where anchors have failed per Figure 17 in Appendix A.

COST ESTIMATE

The preliminary cost estimate for the structural upgrades delineated above is \$269,000. A detailed breakdown of costs immediately follows. It does not include replacing the metal roof deck, since it is not known if deck replacement will only occur in localized areas where there is deterioration, and furthermore, if the architectural approach will necessitate a heavier deck with increased section properties.

PROJECT: Whittier Peninsula Redevelopment
PROJECT NO: City of Columbus Dept. of Development
41616
PREPARED BY: G.SWEENEY

9/23/2005



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Estimate Unit Cost Summary

Detail Description	Cost	
Truss Reinforcement Detail # 1 & 2	\$	29,237.90
Shear Walls East & West	\$	98,518.45
X - Bracing South Wall	\$	5,706.04
New Bracing Along Column Line 12	\$	5,412.24
Shear Wall North	\$	27,769.36
Wall North - Shear Transfer Problems	\$	9,613.50
Foundation Detail at North Shear Wall	\$	22,631.60
Foundation Detail at East/West Shear Wall	\$	24,461.50
Wall Anchor to Col. Replacement Detail	\$	10,875.00
Shear Transfer Connection at Shotcrete Rib	\$	5,551.76
Wall Connection at Roof, East & West Sides	\$	27,046.88
Shear Transfer Connection at Shotcrete Rib (North Side)	\$	1,897.76
Total	\$	268,721.98

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 G.SWEENEY 9/23/2005



BURGESS & NIPLE

Estimate Unit Cost

Item Description	Units	Quantity	Cost	Total
Truss Reinforcement Detail # 1 & 2 W10 x 15 - 20' (14 ea.)	lf	280	30.00	8,400
Truss Reinforcement Detail # 1 & 2 C 3x5 - 6"	ea	56	25.00	1,400
Truss Reinforcement Detail # 1 & 2 Angle 2 1/2" x 2 1/2" x 1/4" x (231 lf.)	lbs	947	1.81	1,714
Truss Reinforcement Detail # 1 & 2 Corner angle plate	ea	28	50.00	1,400
Truss Reinforcement Detail # 1 & 2 Angle 1 3/4" x 1 3/4" x 1/4" x (392 lf 0	lbs	1250.0	1.81	2,263
Truss Reinforcement Detail # 1 & 2 Welding 3/16"	lf	525.0	9.50	4,988

Sub-Total	\$	20,164.07
Bid Margin 10%	\$	2,016.41
Est.w/Incomplete doc. & or Drawings 20%	\$	4,032.81
Construction Cont.5%	\$	1,008.20
General Cond. 10%	\$	2,016.41
Total	\$	29,237.90

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BURGESS & NIPLE

Estimate Unit Cost

Item Description	Units	Quantity	Cost	Total
Shear Walls East & West Saw Cut CMU Walls	lf	1107.0	2.50	2,768
Shear Walls East & West Resteel #3 & #4	lb	4191.0	1.12	4,677
Shear Walls East & West Wall Fasteners for Rebar	ea	3417.0	1.50	5,126
Shear Walls East & West Lift or Scaffolding	ls	1.0	3,325.00	3,325
Shear Walls East & West Shotcrete	sf	3067.0	14.00	42,938
Shear Walls East & West Angle 4 x 3 x 1/4" x (119 lf)	lbs	690.0	1.81	1,249
Shear Walls East & West Welding 1/4"	lf	119.0	14.30	1,702
Shear Walls East & West Adhesive Anchors	ea	176.0	35.00	6,160

Sub-Total	\$	67,943.76
Bid Margin 10%	\$	6,794.38
Est.w/Incomplete doc. & or Drawings 20%	\$	13,588.75
Construction Cont.5%	\$	3,397.19
General Cond. 10%	\$	6,794.38
Total	\$	98,518.45

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BURGESS & NIPLE

Estimate Unit Cost

Item Description	Units	Quantity	Cost	Total
X - Bracing South Wall Demo Existing Bracing	ls	1.0	300.00	300
X - Bracing South Wall Angle 3 1/2" x 3 1/2" x 5/16"	lbs	1440.0	1.81	2,606
X - Bracing South Wall Corner Plates	ea	16.0	50.00	800
X - Bracing South Wall Weld	lf	16.0	14.30	229

Sub-Total	\$	3,935.20
Bid Margin 10%	\$	393.52
Est.w/Incomplete doc. & or Drawings 20%	\$	787.04
Construction Cont.5%	\$	196.76
General Cond. 10%	\$	393.52
Total	\$	5,706.04

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Estimate Unit Cost

Item Description	Units	Quantity	Cost	Total
New Bracing Along Column Line 12 Angle 4" x 4" x 1/4"	lbs	634.0	1.81	1,148
New Bracing Along Column Line 12 Angle 6" x 4" x 7/16"	lbs	1144.0	1.81	2,071
New Bracing Along Column Line 12 Corner Plates	ea	8.0	50.00	400
New Bracing Along Column Line 12 Weld	lf	8.0	14.30	114

Sub-Total	\$	3,732.58
Bid Margin 10%	\$	373.26
Est.w/Incomplete doc. & or Drawings 20%	\$	746.52
Construction Cont.5%	\$	186.63
General Cond. 10%	\$	373.26
Total	\$	5,412.24

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Estimate Unit Cost

Item Description	Units	Quantity	Cost	Total
Shear Wall North Saw Cut CMU Walls	lf	383.0	2.50	958
Shear Wall North Resteel #3 & #4	lb	1330.0	1.12	1,484
Shear Wall North Wall Fasteners for Rebar	ea	865.0	1.50	1,298
Shear Wall North Lift or Scaffolding	ls	1.0	2,000.00	2,000
Shear Wall North Shotcrete	sf	958.0	14.00	13,412

Sub-Total	\$	19,151.28
Bid Margin 10%	\$	1,915.13
Est.w/Incomplete doc. & or Drawings 20%	\$	3,830.26
Construction Cont.5%	\$	957.56
General Cond. 10%	\$	1,915.13
Total	\$	27,769.36

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Estimate Unit Cost

Item Description	Units	Quantity	Cost	Total
Wall North - Shear Transfer Problems MC 6 x 12 - 20' (6 EA.)	lf	120.0	45.00	5,400
Wall North - Shear Transfer Problems Adhesive Anchors	ea	18.0	35.00	630
Wall North - Shear Transfer Problems Lift or Scaffolding	ls	1.0	600.00	600

Sub-Total	\$	6,630.00
Bid Margin 10%	\$	663.00
Est.w/Incomplete doc. & or Drawings 20%	\$	1,326.00
Construction Cont.5%	\$	331.50
General Cond. 10%	\$	663.00
Total	\$	9,613.50

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Estimate Unit Cost

Item Description	Units	Quantity	Cost	Total
Foundation Detail at North Shear Wall Saw Cut Concrete Floor	lf	84.0	9.50	798
Foundation Detail at North Shear Wall Demo Concrete Floor Slab	cy	6.0	150.00	900
Foundation Detail at North Shear Wall Excavation	cy	30.0	35.00	1,050
Foundation Detail at North Shear Wall Granular Backfill	cy	30.0	35.00	1,050
Foundation Detail at North Shear Wall Haul Waste Soil	cy	30.0	12.00	360
Foundation Detail at North Shear Wall Concrete Slab on Grade	cy	6.0	350.00	2,100
Foundation Detail at North Shear Wall Concrete at Foundation Wall w/Reinforcement	cy	7.0	650.00	4,550
Foundation Detail at North Shear Wall Dowels 18"	ea	80.0	5.00	400
Foundation Detail at North Shear Wall Drill Concrete 7" Embed	ea	80.0	15.00	1,200
Foundation Detail at North Shear Wall Adhesive Set	ea	80.0	40.00	3,200

Sub-Total \$ 15,608.00

Bid Margin 10% \$ 1,560.80

Est.w/Incomplete doc. & or Drawings 20% \$ 3,121.60

Construction Cont.5% \$ 780.40

General Cond. 10% \$ 1,560.80

Total \$ 22,631.60

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Estimate Unit Cost

Item Description	Units	Quantity	Cost	Total
Foundation Detail at East/West Shear Wall Excavation	cy	20.0	35.00	700
Foundation Detail at East/West Shear Wall Granular Backfill	cy	20.0	35.00	700
Foundation Detail at East/West Shear Wall Haul Waste Soil	cy	20.0	12.00	240
Foundation Detail at East/West Shear Wall Concrete at Foundation Wall w/Reinforcement	cy	7.0	650.00	4,550
Foundation Detail at East/West Shear Wall Dowels 18"	ea	178.0	5.00	890
Foundation Detail at East/West Shear Wall Drill Concrete 7" Embed	ea	178.0	15.00	2,670
Foundation Detail at East/West Shear Wall Adhesive Set	ea	178.0	40.00	7,120

Sub-Total	\$	16,870.00
Bid Margin 10%	\$	1,687.00
Est.w/Incomplete doc. & or Drawings 20%	\$	3,374.00
Construction Cont.5%	\$	843.50
General Cond. 10%	\$	1,687.00
Total	\$	24,461.50

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BURGESS & NIPLE

Estimate Unit Cost

Item Description	Units	Quantity	Cost	Total
Wall Anchor to Col. Replacement Detail Bent Plate 1/4" x 2" x 0.6"	ea	50.0	15.00	750
Wall Anchor to Col. Replacement Detail Adhesive Anchors	ea	100.0	35.00	3,500
Wall Anchor to Col. Replacement Detail Angle 4" x 3" x 1/4" x 0.2"	ea	50.0	35.00	1,750
Wall Anchor to Col. Replacement Detail Lift or Scaffolding	ea	1.0	1,500.00	1,500

Sub-Total	\$	7,500.00
Bid Margin 10%	\$	750.00
Est.w/Incomplete doc. & or Drawings 20%	\$	1,500.00
Construction Cont.5%	\$	375.00
General Cond. 10%	\$	750.00
Total	\$	10,875.00

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BURGESS & NIPLE

Estimate Unit Cost

Item Description	Units	Quantity	Cost	Total
Shear Transfer Connection at Shotcrete Rib Plate 3/8" x 0.22" x 0.8" w/Headed Studs @ 6"	ea	12.0	25.00	300
Shear Transfer Connection at Shotcrete Rib MC 6 x 12 - 20' (3 EA.)	ea	60.0	45.00	2,700
Shear Transfer Connection at Shotcrete Rib Weld 1/4"	ea	16.0	14.30	229
Shear Transfer Connection at Shotcrete Rib Lift or Scaffolding	ls	1.0	600.00	600

Sub-Total	\$	3,828.80
Bid Margin 10%	\$	382.88
Est.w/Incomplete doc. & or Drawings 20%	\$	765.76
Construction Cont.5%	\$	191.44
General Cond. 10%	\$	382.88
Total	\$	5,551.76

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BURGESS & NIPLE

Estimate Unit Cost

Item Description	Units	Quantity	Cost	Total
Wall Connection at Roof, East & West Sides Angle 5" x 3" x 1/4" (620 lf)	lb	4092.0	1.81	7,407
Wall Connection at Roof, East & West Sides Adhesive Anchors	ea	207.0	35.00	7,245
Wall Connection at Roof, East & West Sides Weld 1/4"	lf	105.0	14.30	1,502
Shear Transfer Connection at Shotcrete Rib Lift or Scaffolding	ls	1.0	2,500.00	2,500

Sub-Total	\$	18,653.02
Bid Margin 10%	\$	1,865.30
Est.w/Incomplete doc. & or Drawings 20%	\$	3,730.60
Construction Cont.5%	\$	932.65
General Cond. 10%	\$	1,865.30
Total	\$	27,046.88

PROJECT: Whittier Structural Analysis
 PROJECT NO: City of Columbus Dept. of Development
 PREPARED BY: 41616
 G.SWEENEY 9/23/2005



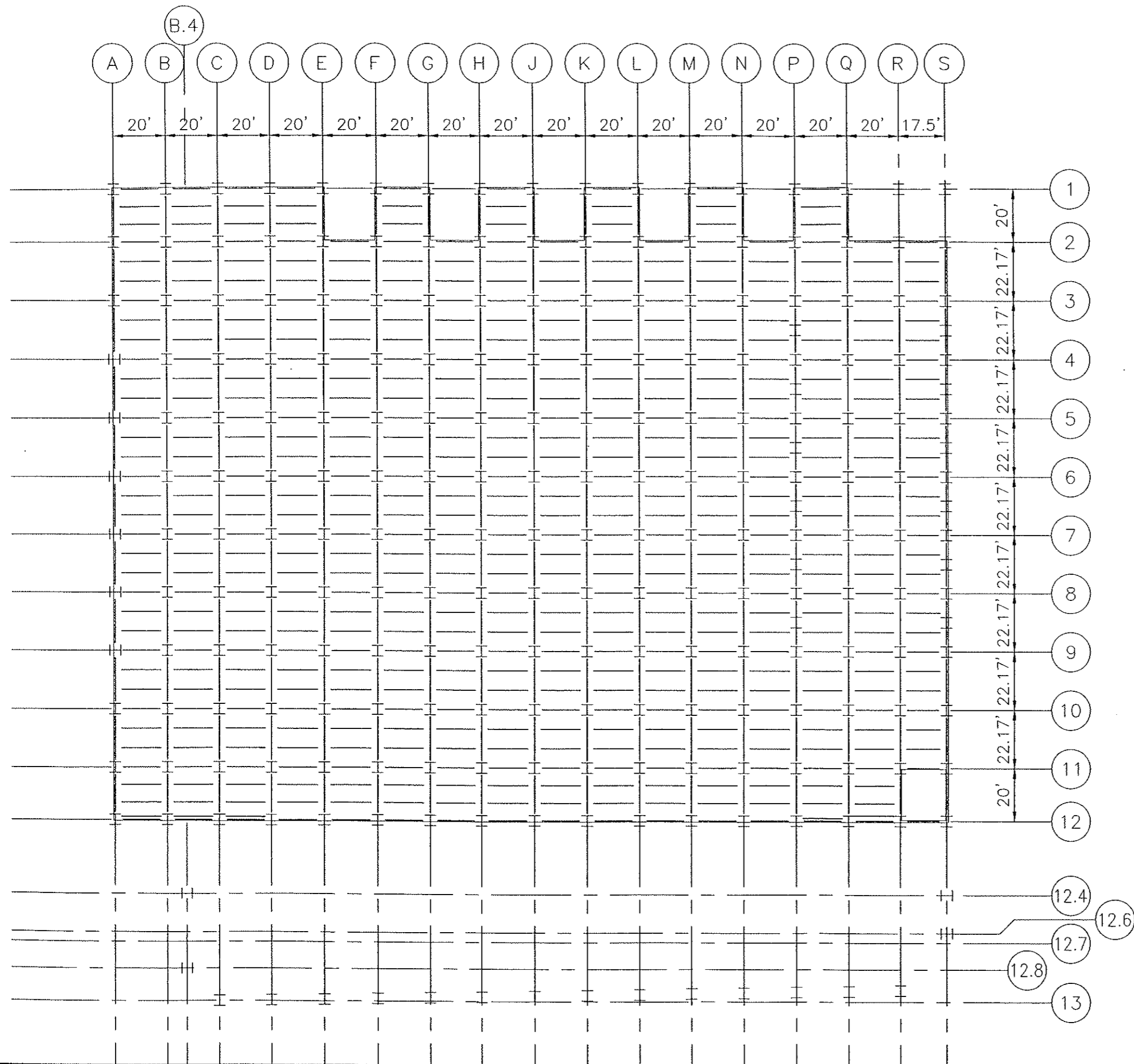
BURGESS & NIPLE

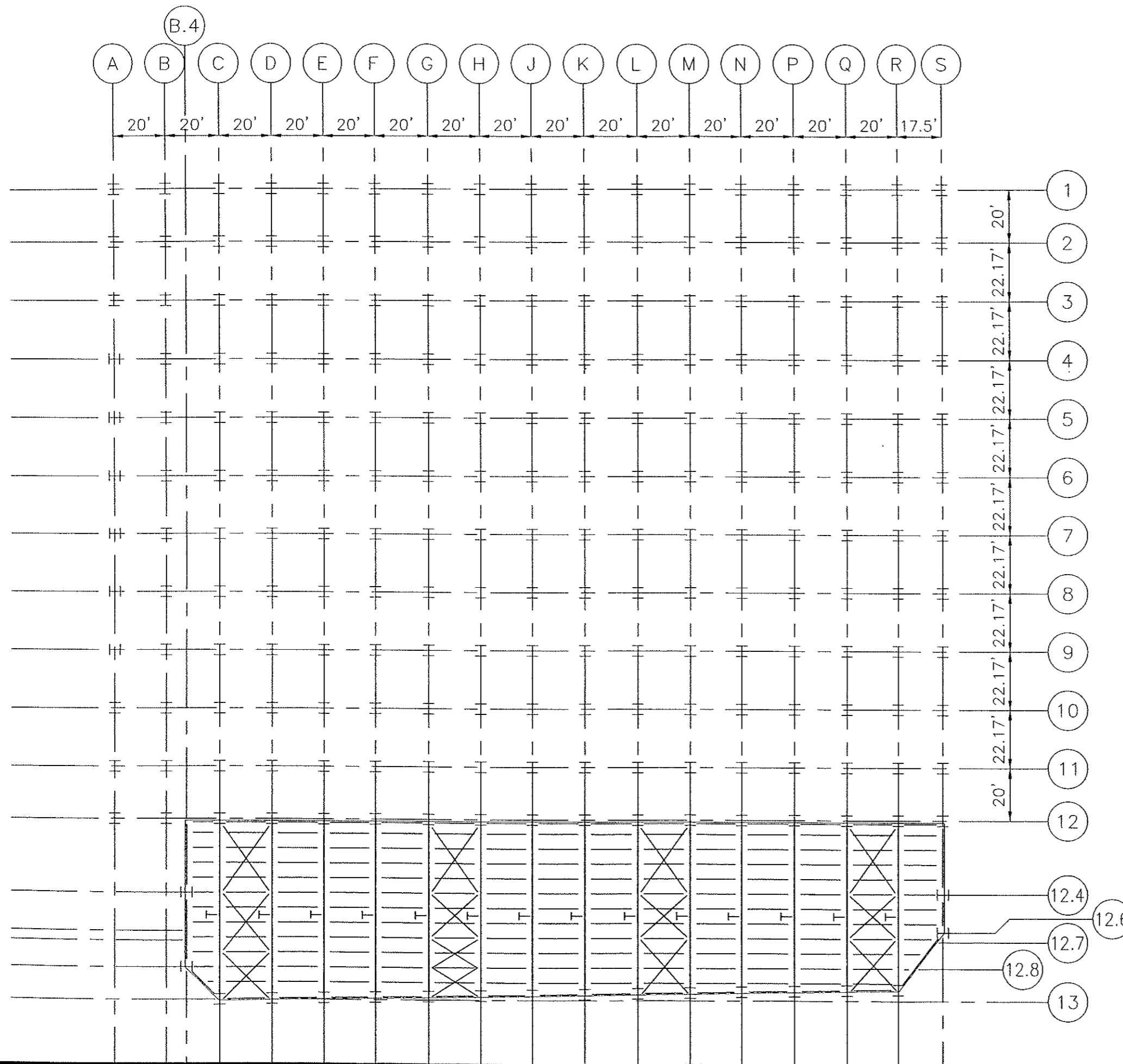
Estimate Unit Cost

Item Description	Units	Quantity	Cost	Total
Shear Transfer Connection at Shotcrete Rib (North Side) Plate 3/8" x 0.22" x 0.8" w/Headed Studs @ 6"	ea	12.0	25.00	300
Shear Transfer Connection at Shotcrete Rib (North Side) Angle 3" x 3" x 1/4" x 3"	ea	12.0	15.00	180
Shear Transfer Connection at Shotcrete Rib (North Side) Weld 1/4"	ea	16.0	14.30	229
Shear Transfer Connection at Shotcrete Rib (North Side) Lift or Scaffolding	ls	1.0	600.00	600

Sub-Total	\$	1,308.80
Bid Margin 10%	\$	130.88
Est.w/Incomplete doc. & or Drawings 20%	\$	261.76
Construction Cont.5%	\$	65.44
General Cond. 10%	\$	130.88
Total	\$	1,897.76

Appendix A – Figures



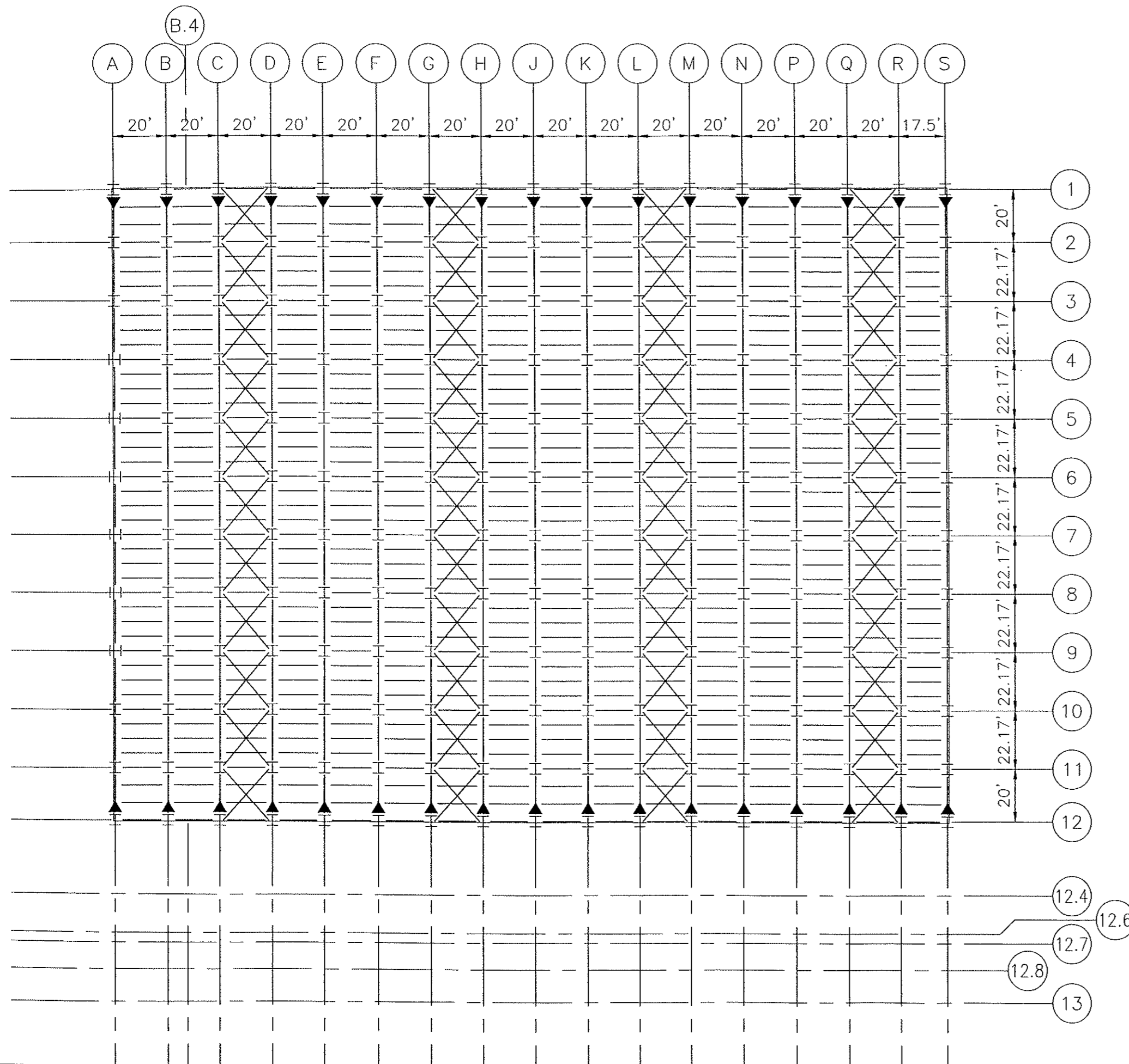


LEGEND


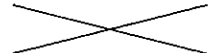
- T TRUSS
- HORIZONTAL BRACING

WHITTIER PENINSULA REDEVELOPMENT
LAZARUS WAREHOUSE

FIGURE 2
LOWER ROOF
SCHEMATIC FRAMING PLAN



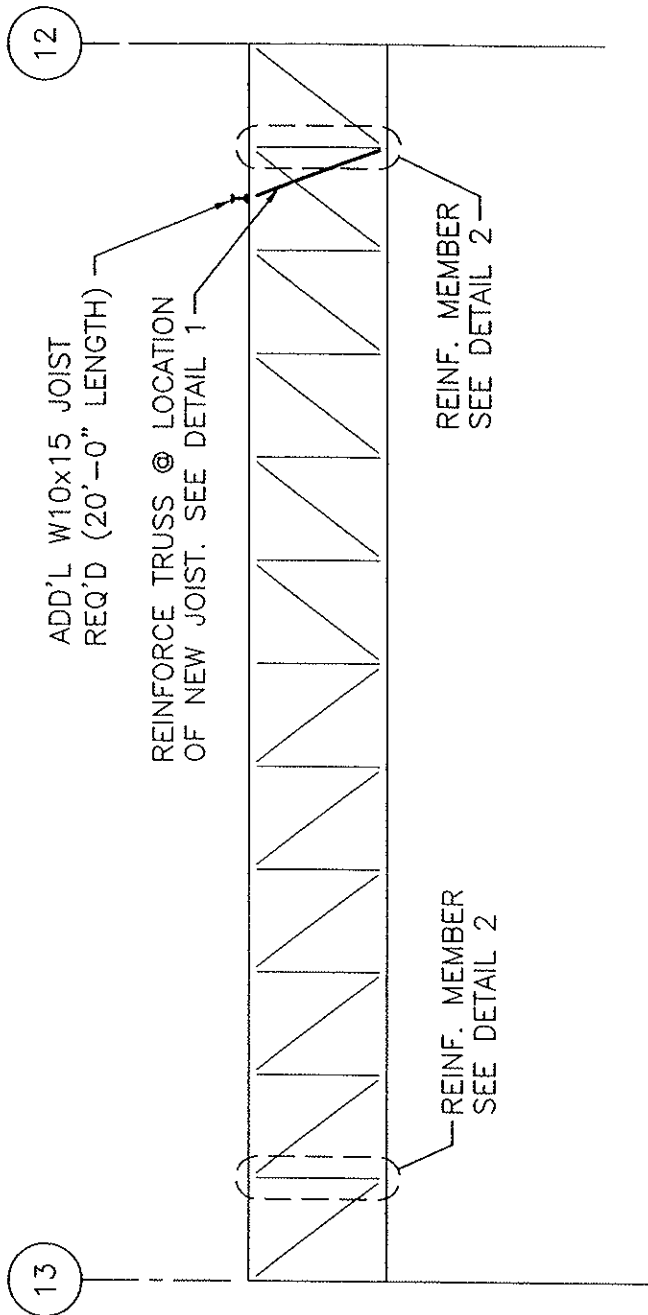
LEGEND

-  MOMENT CONNECTION
-  HORIZONTAL BRACING

WHITTIER PENINSULA REDEVELOPMENT
LAZARUS WAREHOUSE

FIGURE 3

SCHEMATIC ROOF FRAMING PLAN



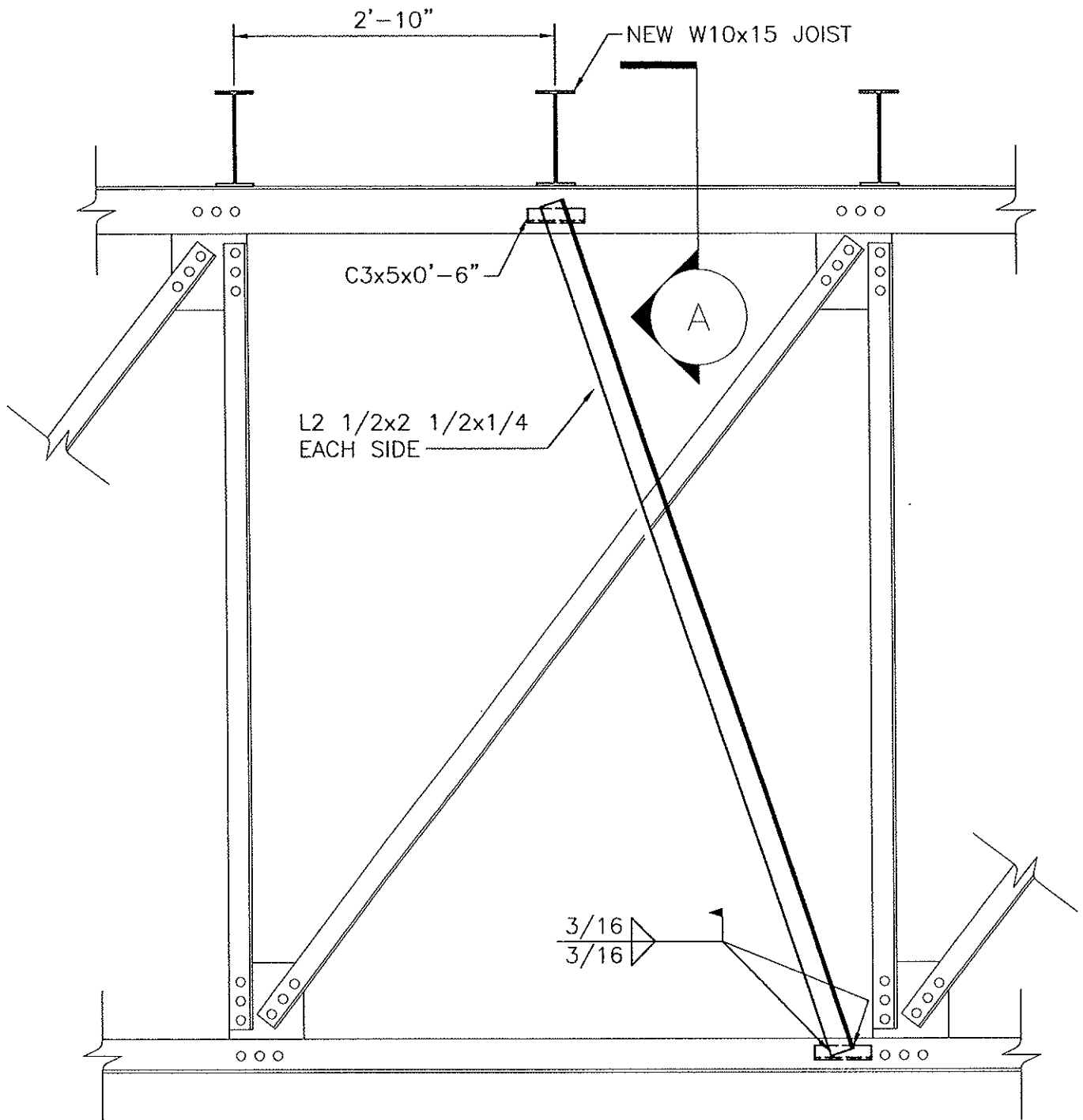
TYPICAL TRUSS IN SOUTH BAY – LOOKING WEST (TOTAL OF 14)

SCALE: 3/16" = 1'-0"

WHITTIER PENINSULA REDEVELOPMENT
LAZARUS WAREHOUSE

FIGURE 4

TYPICAL TRUSS IN SOUTH BAY
LOOKING WEST



2

TRUSS REINFORCEMENT DETAIL

SCALE: $\frac{3}{4}" = 1'-0"$

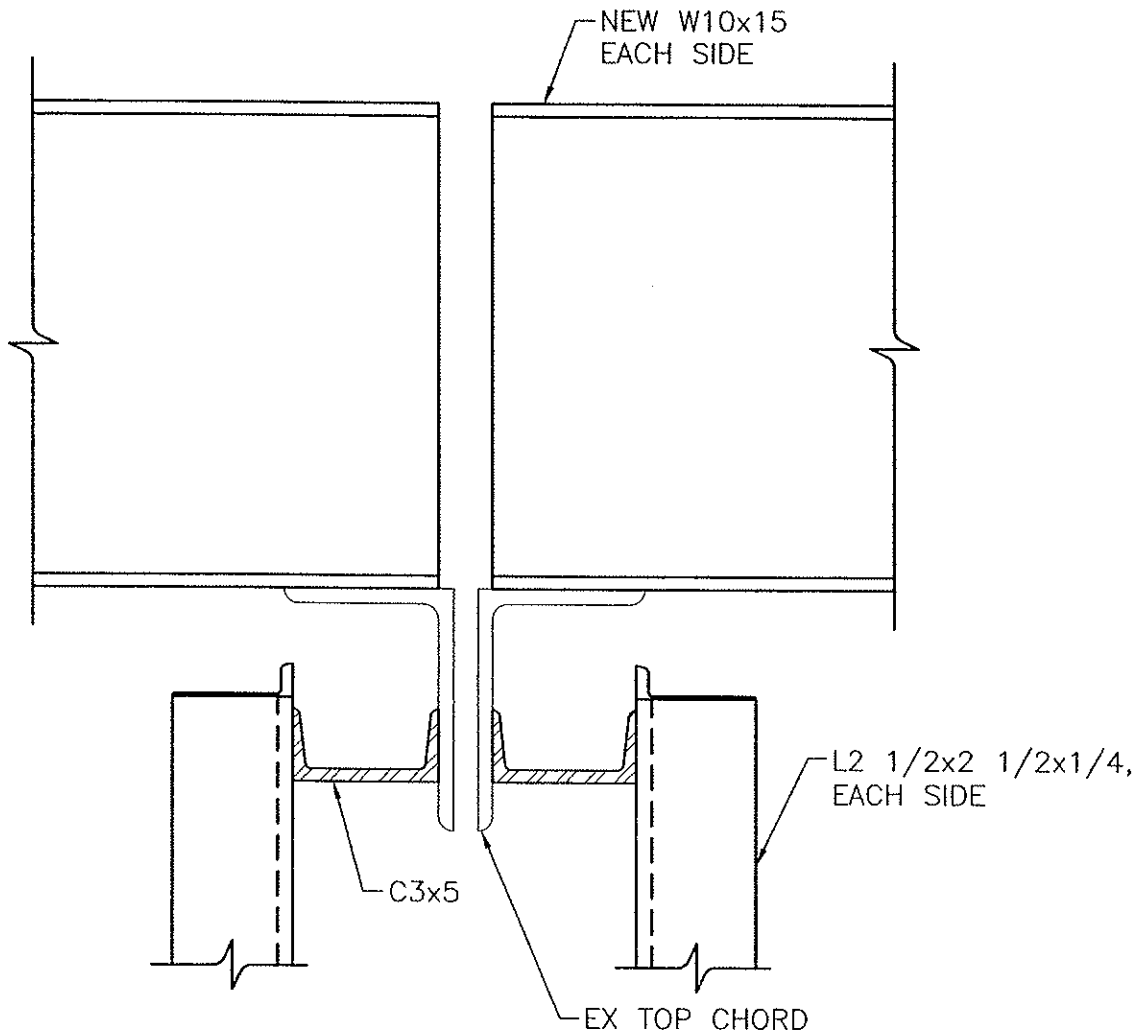
WHITTIER PENINSULA REDEVELOPMENT
LAZARUS WAREHOUSE

FIGURE 5

TRUSS REINFORCEMENT DETAIL

BURGESS & NIPLE, INC
ENGINEERS AND ARCHITECTS

SEPTEMBER 2005



A

SECTION

SCALE: 3" = 1'-0"

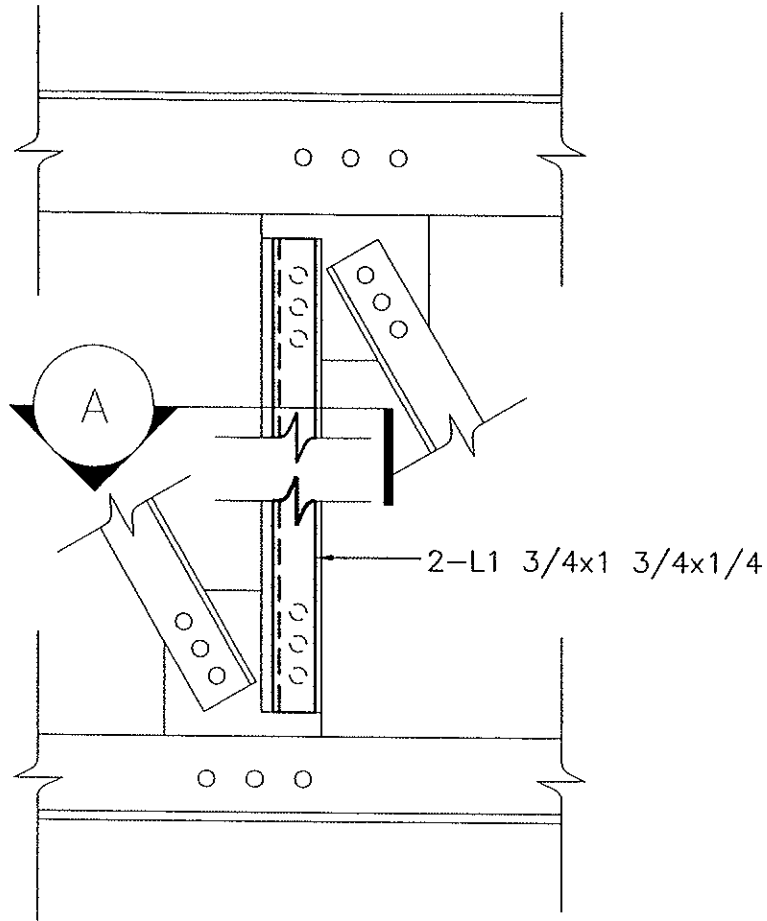
WHITTIER PENINSULA REDEVELOPMENT
LAZARUS WAREHOUSE

FIGURE 5A

TRUSS REINFORCEMENT DETAIL
SECTION A

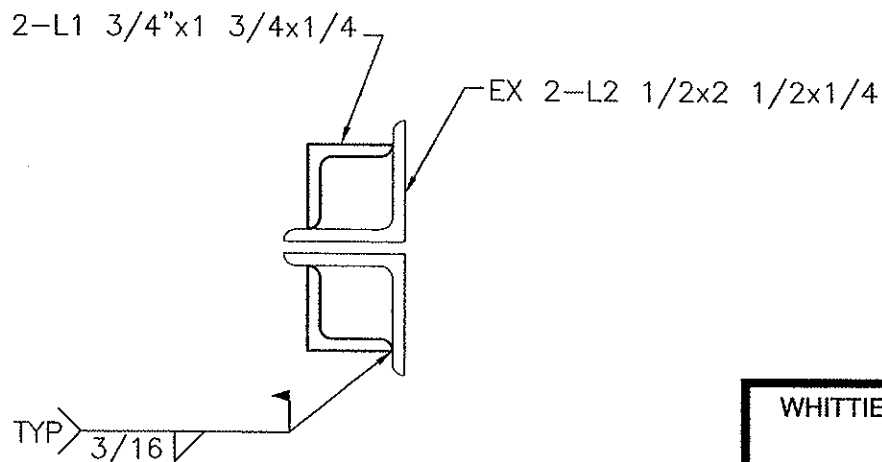
BURGESS & NIPLE, INC
ENGINEERS AND ARCHITECTS

SEPTEMBER 2005



2 TRUSS VERTICAL REINFORCEMENT DETAIL

SCALE: 1 1/2" = 1'-0"



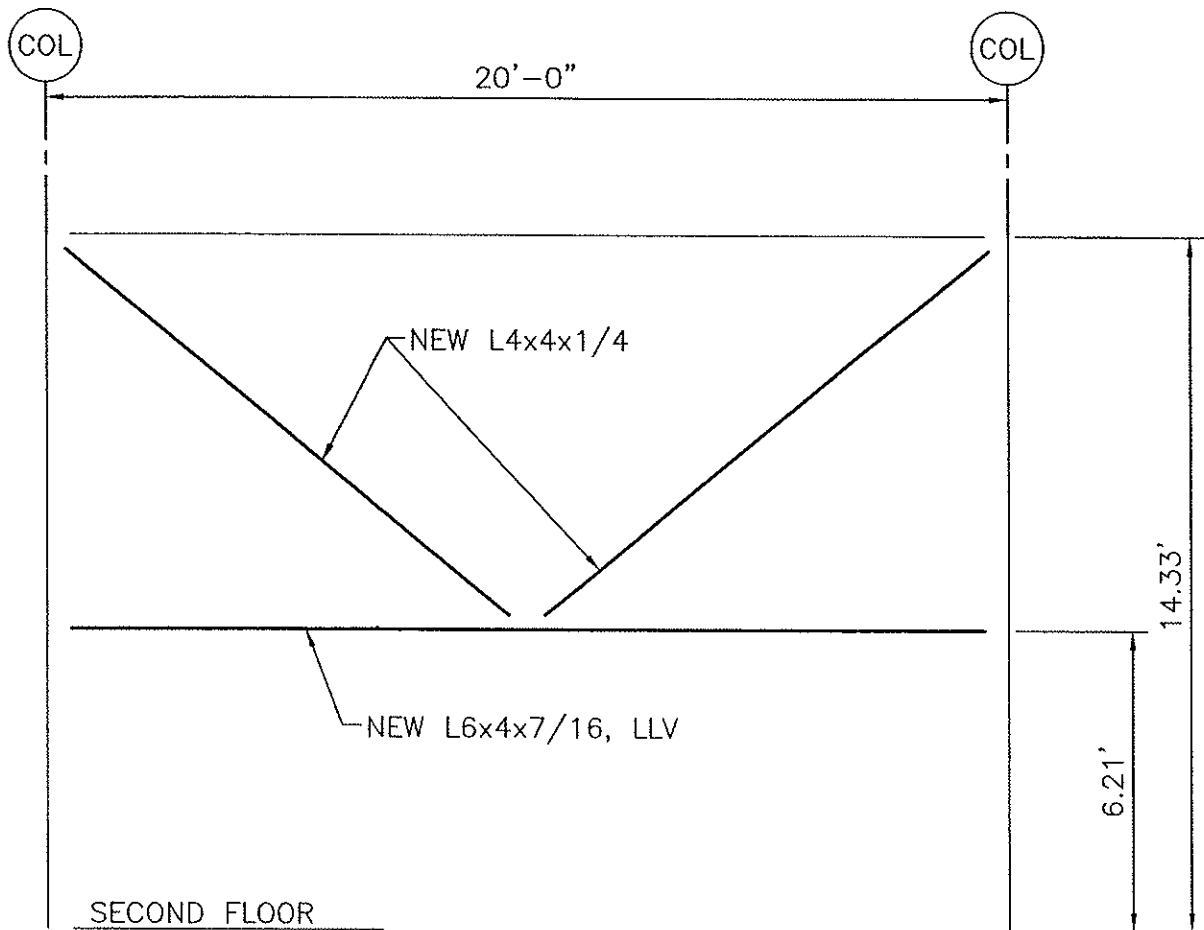
A SECTION

SCALE: 3" = 1'-0"

WHITTIER PENINSULA REDEVELOPMENT
LAZARUS WAREHOUSE

FIGURE 6

TRUSS VERTICAL
REINFORCEMENT DETAIL



NOTE: FIELD VERIFY ALL DIMENSIONS.

NEW BRACING ALONG COLUMN LINE 12 (TYP OF 4 BAYS)

SCALE: 1/4" = 1'-0"

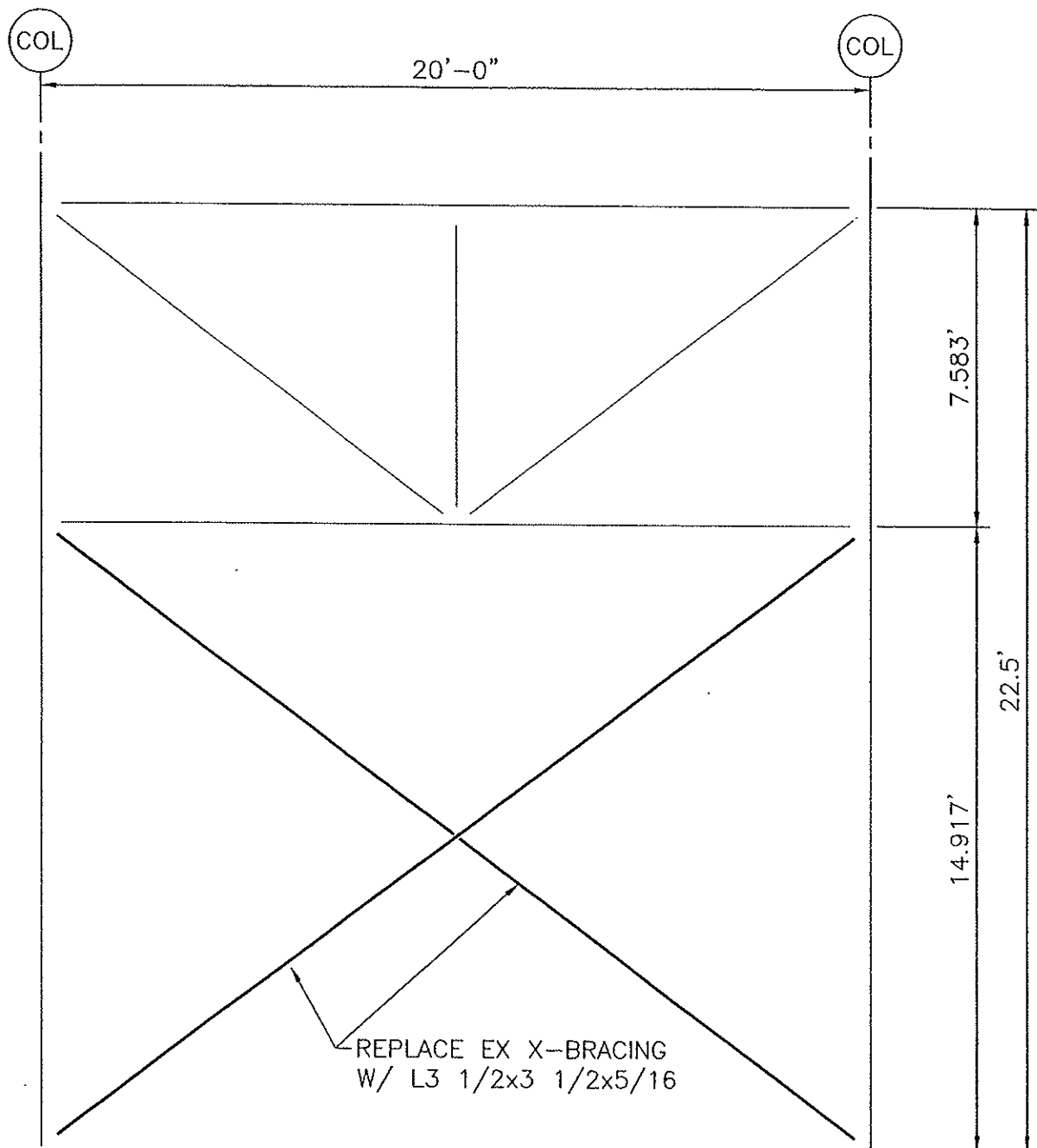
WHITTIER PENINSULA REDEVELOPMENT
LAZARUS WAREHOUSE

FIGURE 7

NEW BRACING ALONG COLUMN
LINE 12 (TYP OF 4 BAYS)

BURGESS & NIPLE, INC
ENGINEERS AND ARCHITECTS

SEPTEMBER 2005



NOTE: FIELD VERIFY ALL DIMENSIONS.

BRACED BAY ALONG SOUTH WALL (TYP OF 4)

SCALE: 1/4" = 1'-0"

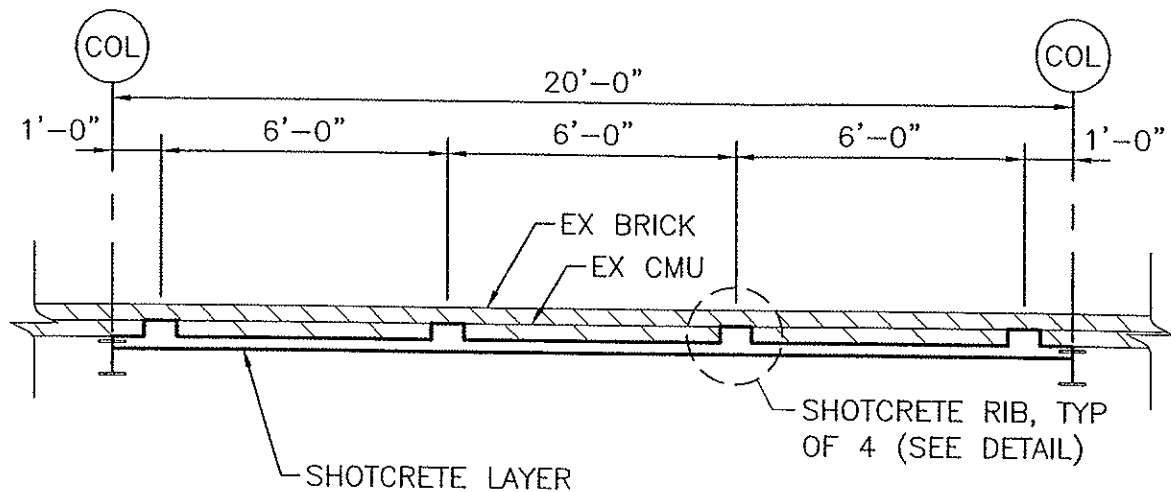
WHITTIER PENINSULA REDEVELOPMENT
LAZARUS WAREHOUSE

FIGURE 8

BRACED BAY ALONG SOUTH
WALL (TYP OF 4)

BURGESS & NIPLE, INC
ENGINEERS AND ARCHITECTS

SEPTEMBER 2005

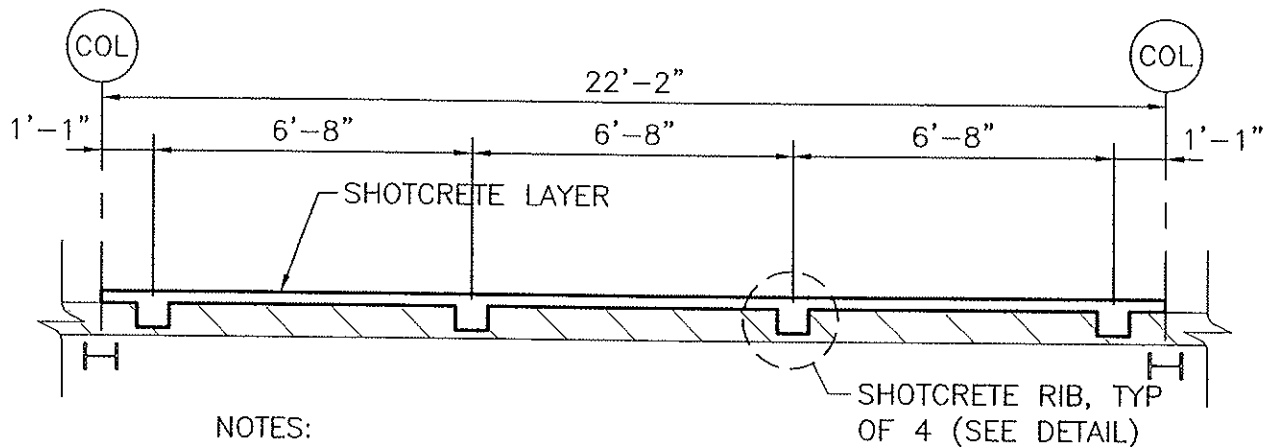


NOTES:

1. REINFORCING NOT SHOWN
2. RIB LAYOUT SIMILAR FOR OTHER BAYS.

TYPICAL BAY WITH SHOTCRETE APPLICATION (1 OF 3 ON NORTH SIDE)

SCALE: $1/4" = 1'-0"$



NOTES:

1. REINFORCING NOT SHOWN
2. RIB LAYOUT SIMILAR FOR OTHER BAYS.

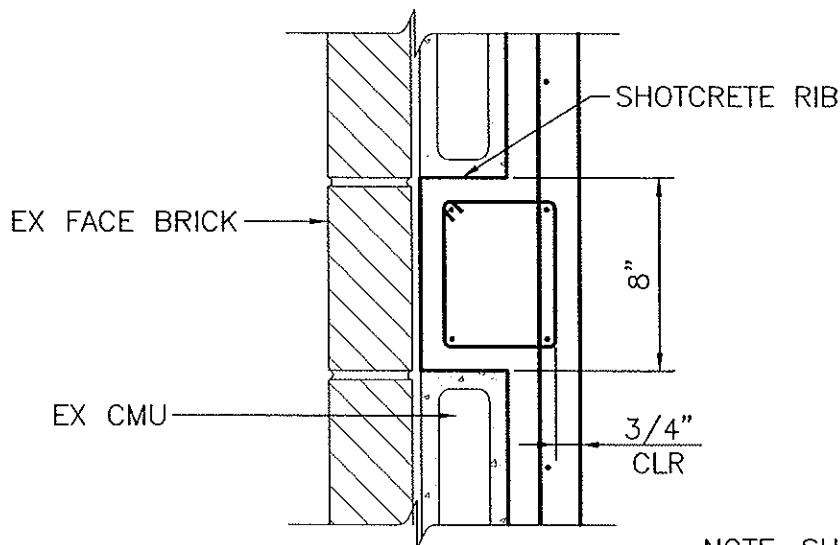
TYPICAL BAY WITH SHOTCRETE APPLICATION (3 BAYS AT EAST AND WEST SIDE—6 TOTAL)

SCALE: $1/4" = 1'-0"$

WHITTIER PENINSULA REDEVELOPMENT
LAZARUS WAREHOUSE

FIGURE 9

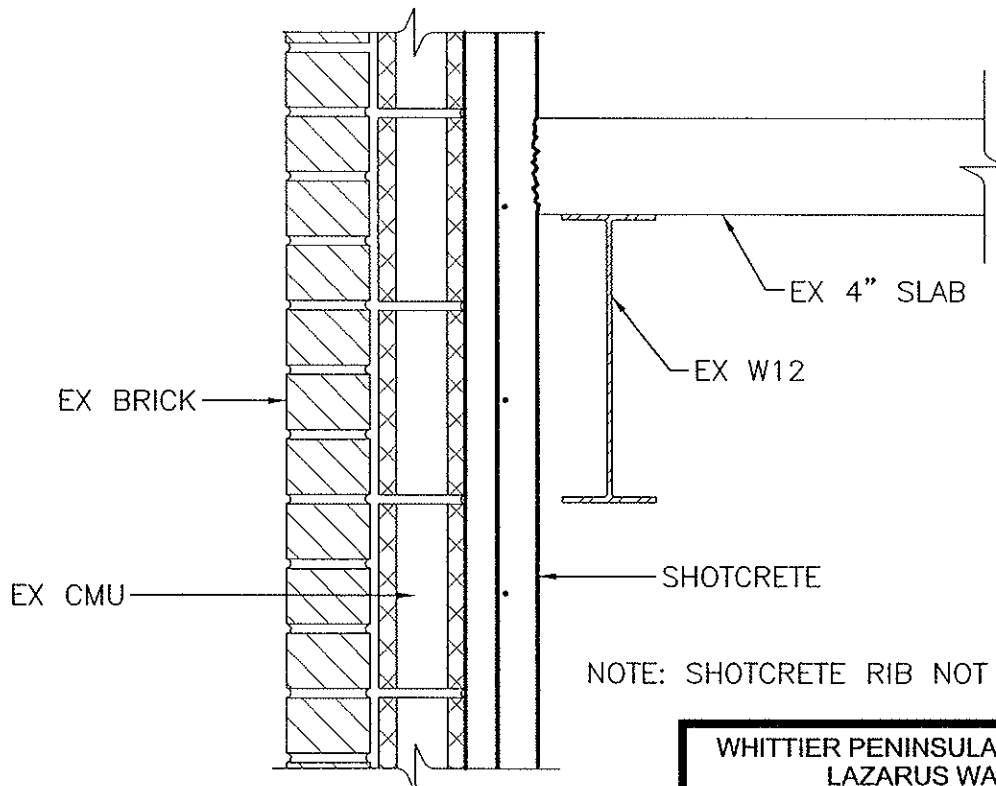
TYPICAL BAY WITH
SHOTCRETE APPLICATION



NOTE: SHEAR TRANSFER CONNECTION
TO FRAMING NOT SHOWN

PLAN OF INTERIOR SHOTCRETE APPLICATION

SCALE: 1 1/2" = 1'-0"



NOTE: SHOTCRETE RIB NOT SHOWN

SECTION OF INTERIOR SHOTCRETE APPLICATION @ SECOND FLOOR

SCALE: 1 1/2" = 1'-0"

WHITTIER PENINSULA REDEVELOPMENT
LAZARUS WAREHOUSE

FIGURE 10
INTERIOR SHOTCRETE
APPLICATION



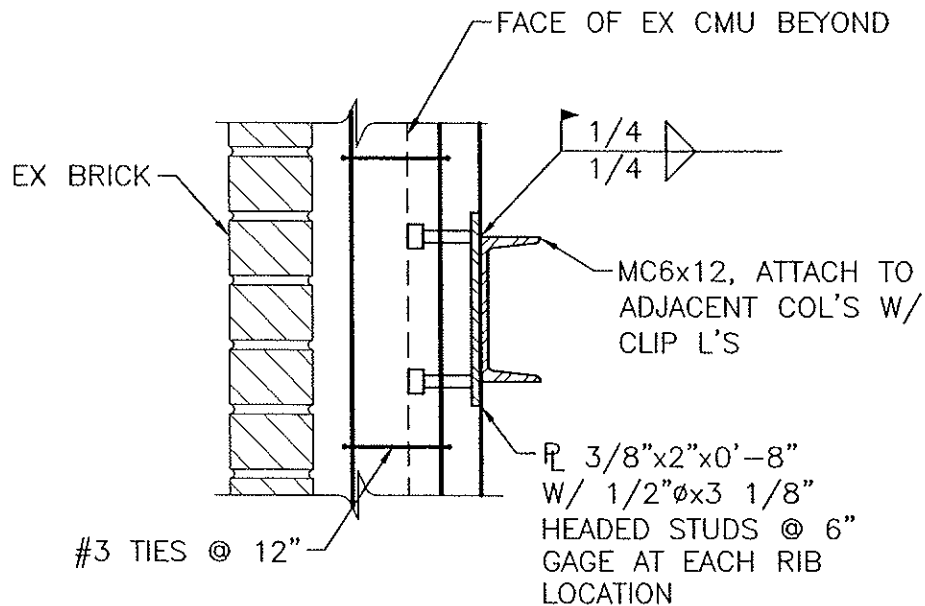
SCALE: 1/16" = 1'-0"



SCALE: 1 1/2" = 1'-0"

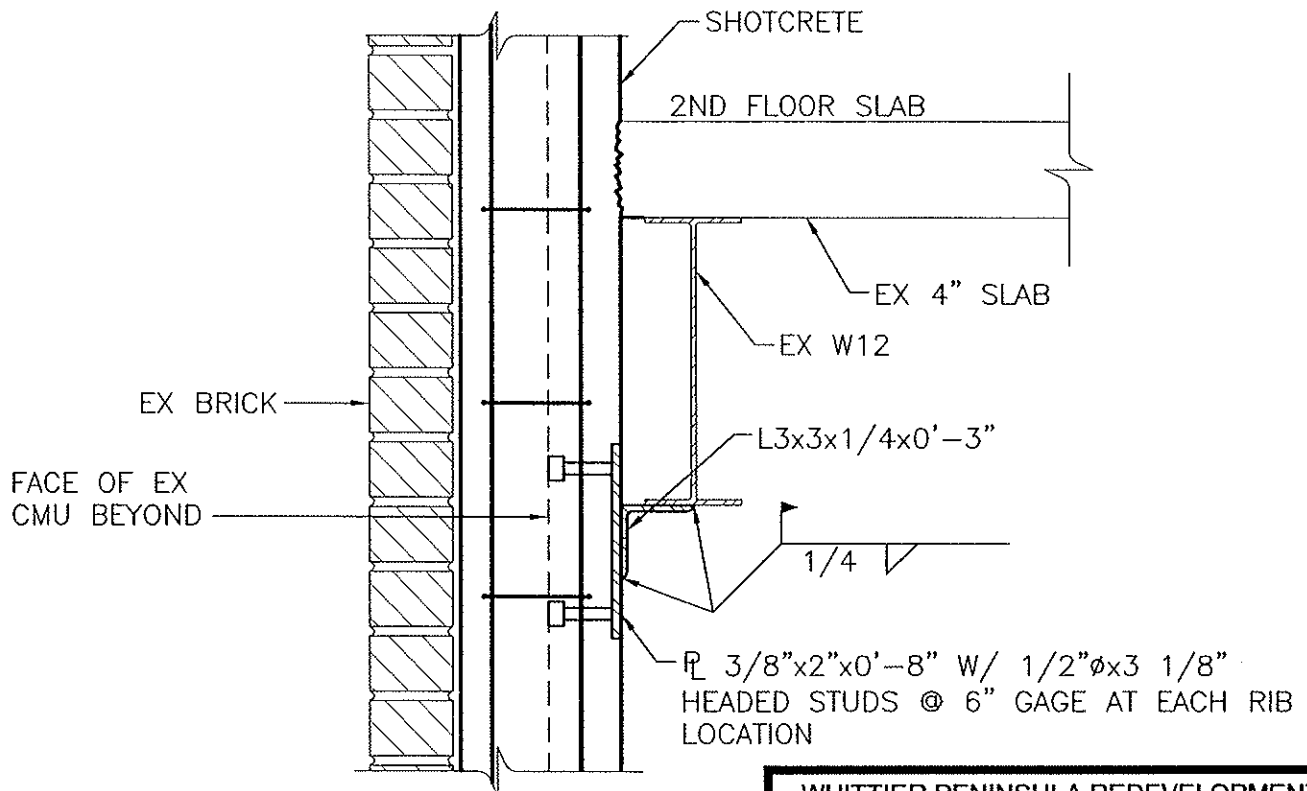
 \triangleleft

SEPTEMBER 2005



SHEAR TRANSFER CONNECTION AT SHOTCRETE RIB (NORTH SIDE)

SCALE: 1 1/2" = 1'-0"



SHEAR TRANSFER CONNECTION AT SHOTCRETE RIB (NORTH SIDE)

SCALE: 1 1/2" = 1'-0"

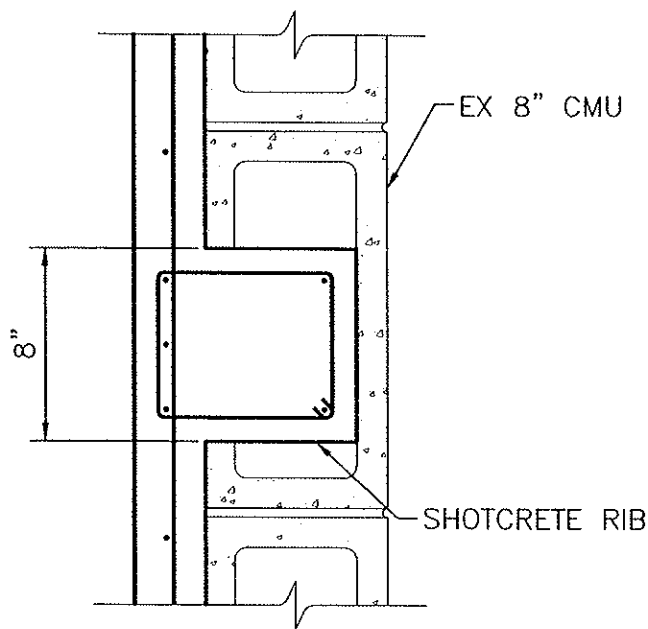
WHITTIER PENINSULA REDEVELOPMENT
LAZARUS WAREHOUSE

FIGURE 12

SHEAR TRANSFER CONNECTION
AT SHOTCRETE RIB

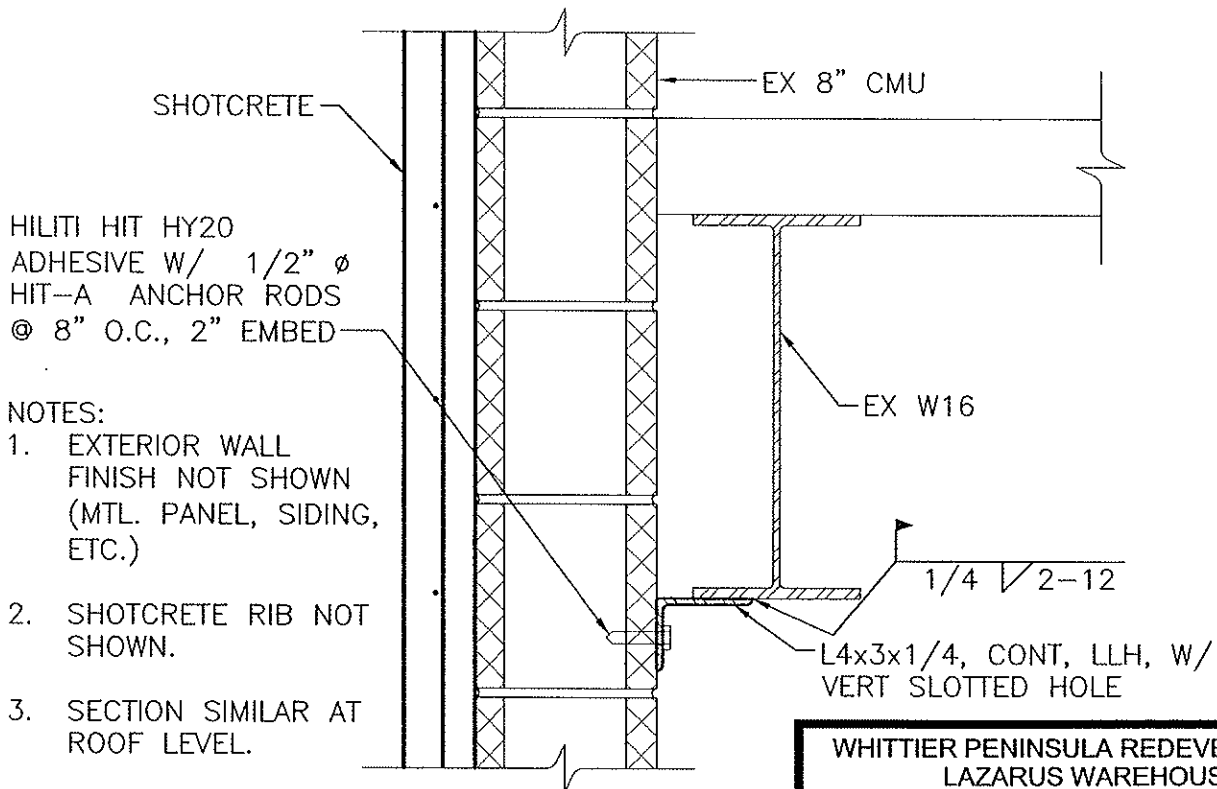
BURGESS & NIPLE, INC
ENGINEERS AND ARCHITECTS

SEPTEMBER 2005



PLAN OF EXTERIOR SHOTCRETE APPLICATION

SCALE: 1 1/2" = 1'-0"



SECTION OF EXTERIOR SHOTCRETE APPLICATION @ SECOND FLOOR

SCALE: 1 1/2" = 1'-0"

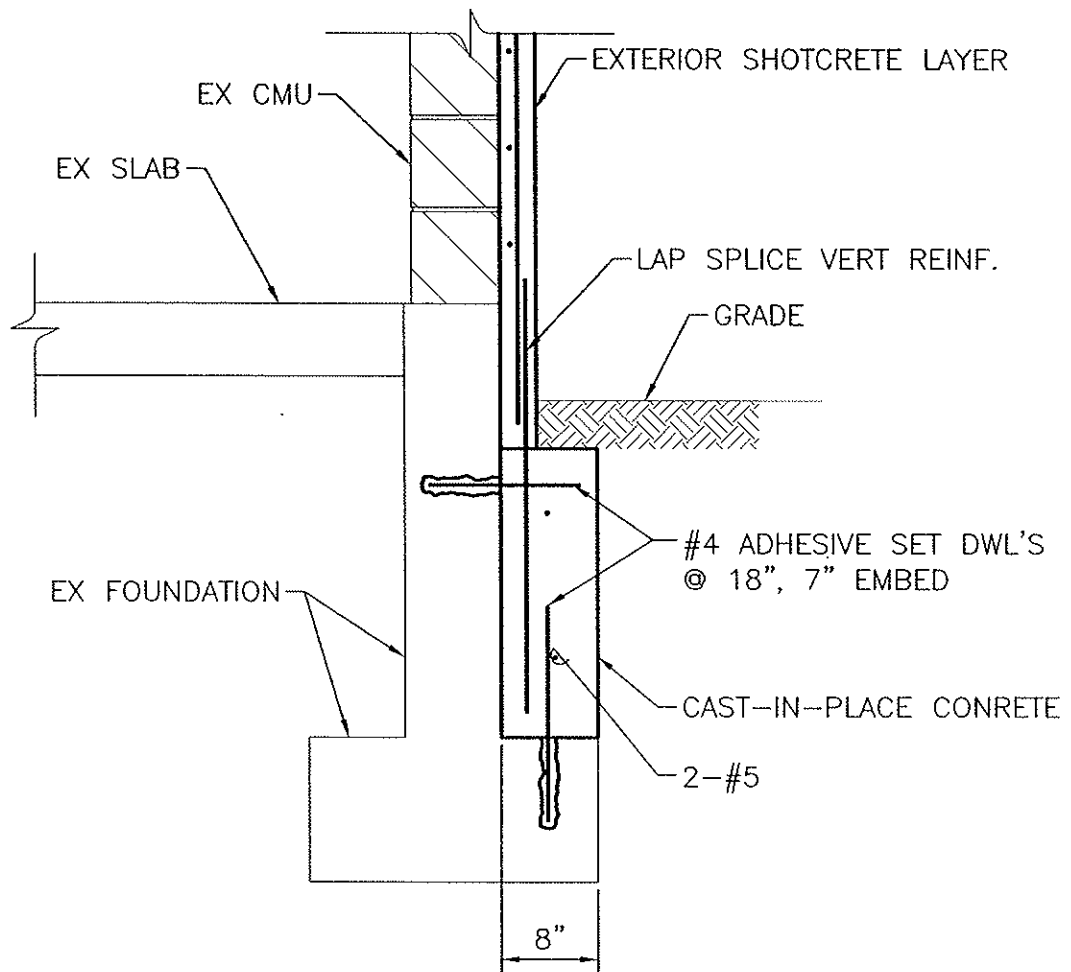
WHITTIER PENINSULA REDEVELOPMENT
LAZARUS WAREHOUSE

FIGURE 13

EXTERIOR SHOTCRETE
APPLICATION

BURGESS & NIPLE, INC
ENGINEERS AND ARCHITECTS

SEPTEMBER 2005



NOTE: FIELD VERIFY EXISTING FOUNDATION CONDITIONS.

FOUNDATION DETAIL AT SHEAR WALLS (EAST AND WEST SIDES)

SCALE: $\frac{3}{4}" = 1'-0"$

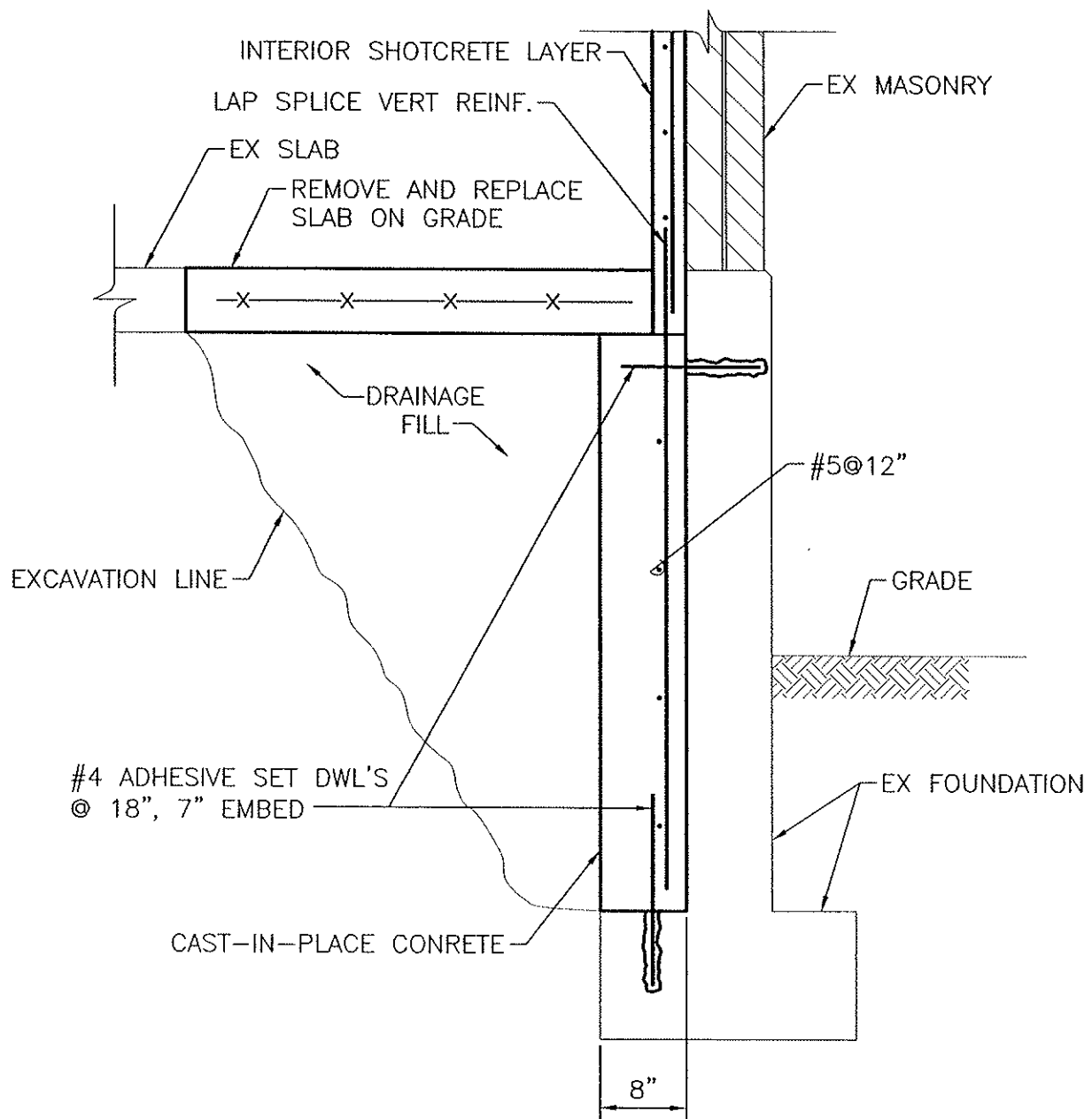
WHITTIER PENINSULA REDEVELOPMENT
LAZARUS WAREHOUSE

FIGURE 14

FOUNDATION AT SHEAR WALLS
(EAST AND WEST SIDES)

BURGESS & NIPLE, INC
ENGINEERS AND ARCHITECTS

SEPTEMBER 2005



NOTE: FIELD VERIFY EXISTING FOUNDATION CONDITIONS.

FOUNDATION DETAIL AT SHEAR WALLS (NORTH SIDE ONLY)

SCALE: 3/4" = 1'-0"

WHITTIER PENINSULA REDEVELOPMENT
LAZARUS WAREHOUSE

FIGURE 15

FOUNDATION AT SHEAR WALLS
(NORTH SIDE ONLY)

BURGESS & NIPLE, INC
ENGINEERS AND ARCHITECTS

SEPTEMBER 2005

L5x3x1/4, LLH, CONT
W/ LONG SLOTTED HOLES

1/4" ∇ 2-12

HILTI HIT HY20 ADHESIVE W/
1/2"Ø HIT-A ANCHOR RODS
@ 3'-0" O.C., 2" EMBED

EX MASONRY

EX BM

WALL CONNECTION AT ROOF (EAST AND WEST SIDES)

SCALE: 1 1/2" = 1'-0"

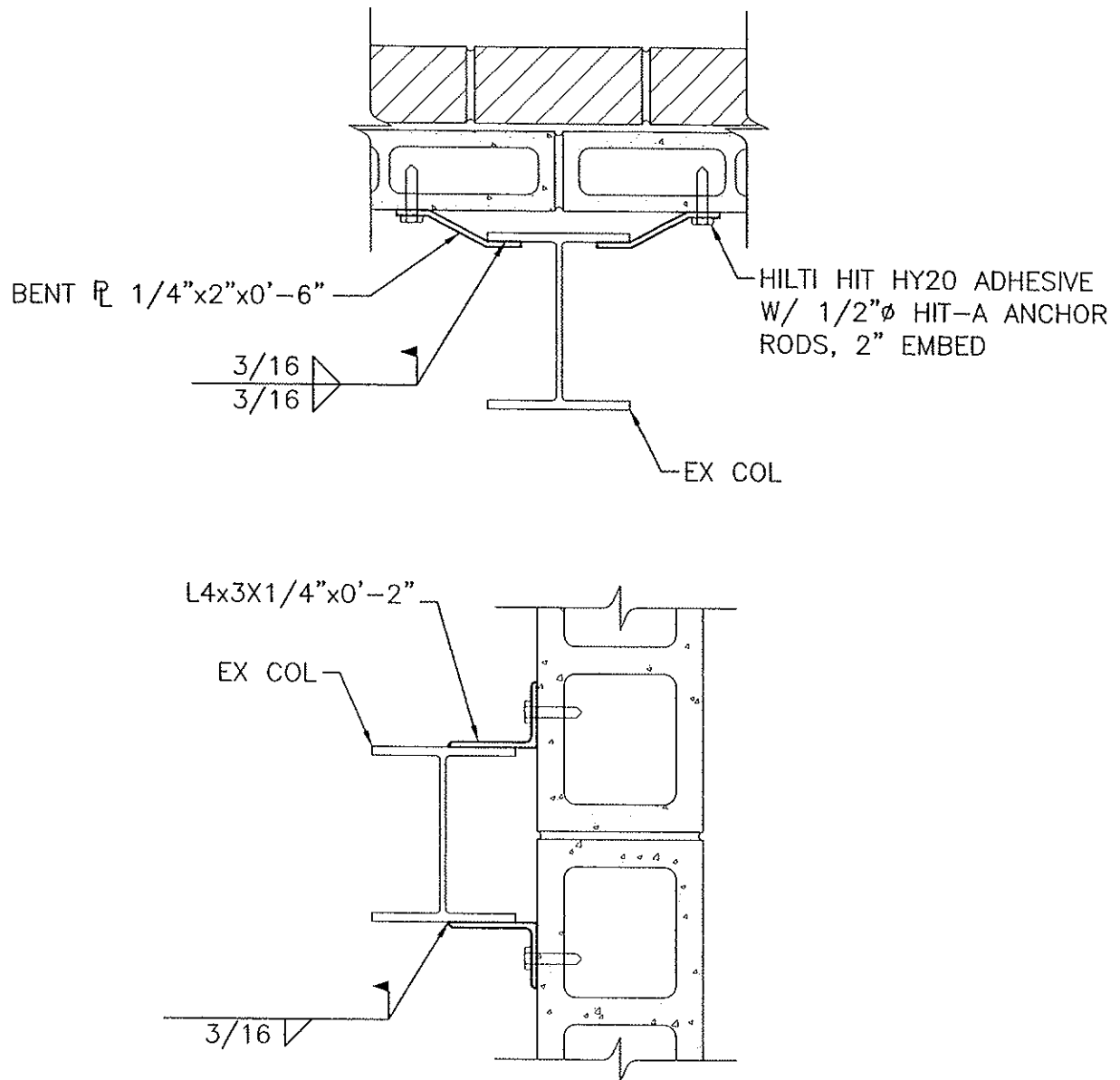
WHITTIER PENINSULA REDEVELOPMENT
LAZARUS WAREHOUSE

FIGURE 16

WALL CONNECTION AT ROOF
(EAST AND WEST SIDES)

BURGESS & NIPLE, INC
ENGINEERS AND ARCHITECTS

SEPTEMBER 2005



WALL ANCHOR TO COLUMN REPLACEMENT DETAIL

SCALE: 1 1/2" = 1'-0"

WHITTIER PENINSULA REDEVELOPMENT
LAZARUS WAREHOUSE

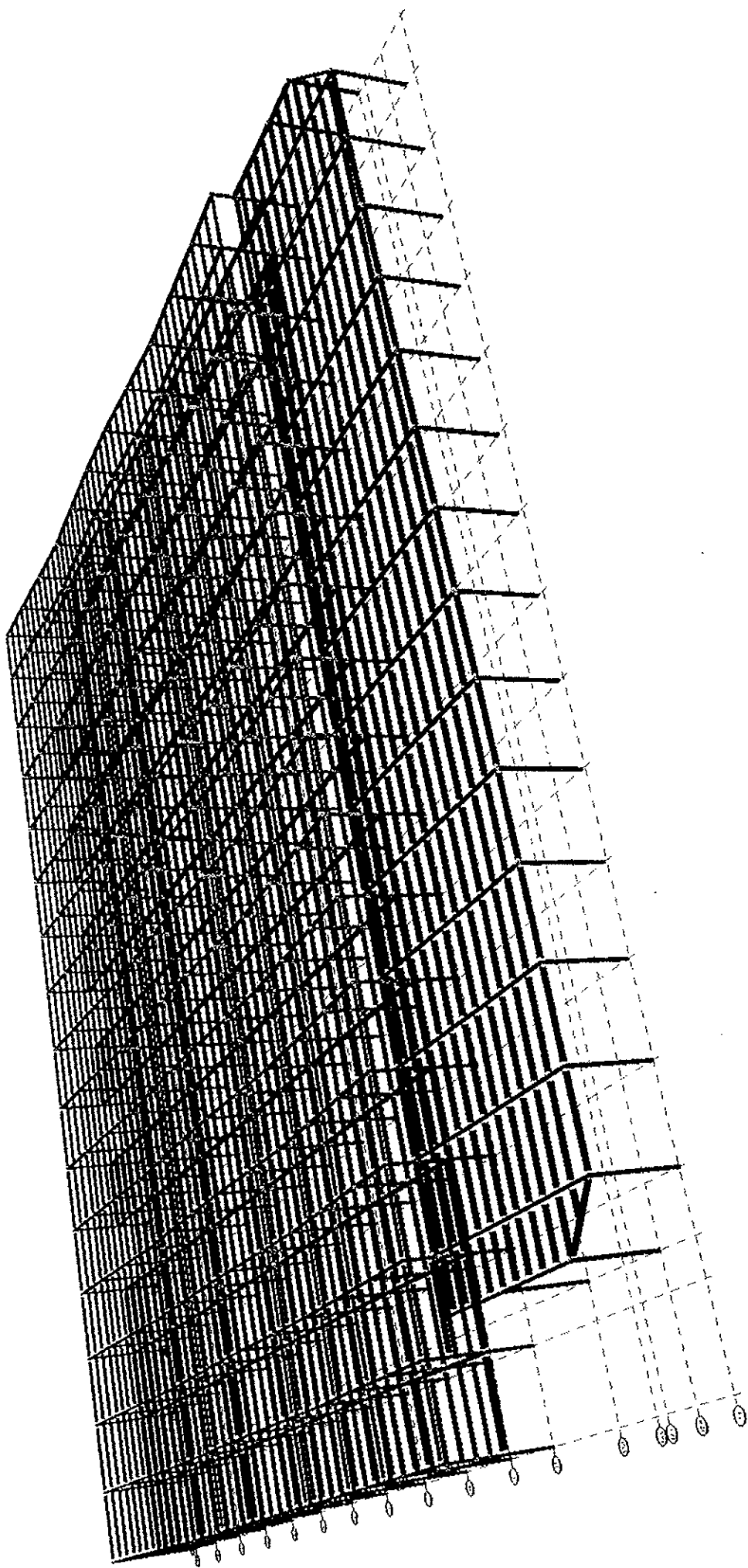
FIGURE 17

WALL ANCHOR TO COLUMN
REPLACEMENT DETAIL

BURGESS & NIPLE, INC
ENGINEERS AND ARCHITECTS

SEPTEMBER 2005

Appendix B – Structural Computer Models





Gravity Beam Design

RAM Steel v8.2
DataBase: WhittierFrame2
Building Code: IBC

09/21/05 19:09:1
Steel Code: ASD 9th Ed

Floor Type: LOWER ROOF

Beam Number = 175

SPAN INFORMATION (ft): I-End (200.00,-5.51) J-End (220.00,-5.51)

Beam Size (User Selected) = W10X12 $F_y = 36.0$ ksi

Total Beam Length (ft) = 20.00

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type
1	0.000	0.000	0.287	0.0%	Roof
	20.000	0.000	0.287		
2	0.000	0.111	0.098	0.0%	Roof
	20.000	0.111	0.098		
3	0.000	0.012	0.000	---	NonR
	20.000	0.012	0.000		

SHEAR: Max V (DL+LL) = 5.08 kips $f_v = 2.71$ ksi $F_v = 14.40$ ksi

MOMENTS:

Span	Cond	Moment kip-ft	@ ft	Lb ft	Cb	Tension Flange fb Fb		Compr Flange fb Fb	
Center	Max +	25.4	10.0	0.0	1.00	27.97	24.00	27.97	24.00
Controlling		25.4	10.0	0.0	1.00	27.97	24.00	---	---

REACTIONS (kips):

	Left	Right
DL reaction	1.23	1.23
Max +LL reaction	3.85	3.85
Max +total reaction	5.08	5.08

DEFLECTIONS:

Dead load (in)	at	10.00 ft =	-0.283	L/D =	847
Live load (in)	at	10.00 ft =	-0.889	L/D =	270
Net Total load (in)	at	10.00 ft =	-1.172	L/D =	205



Software licensed to Burgess

Job Title

Ref

Part

By

Date

Chd

File

Date/Time

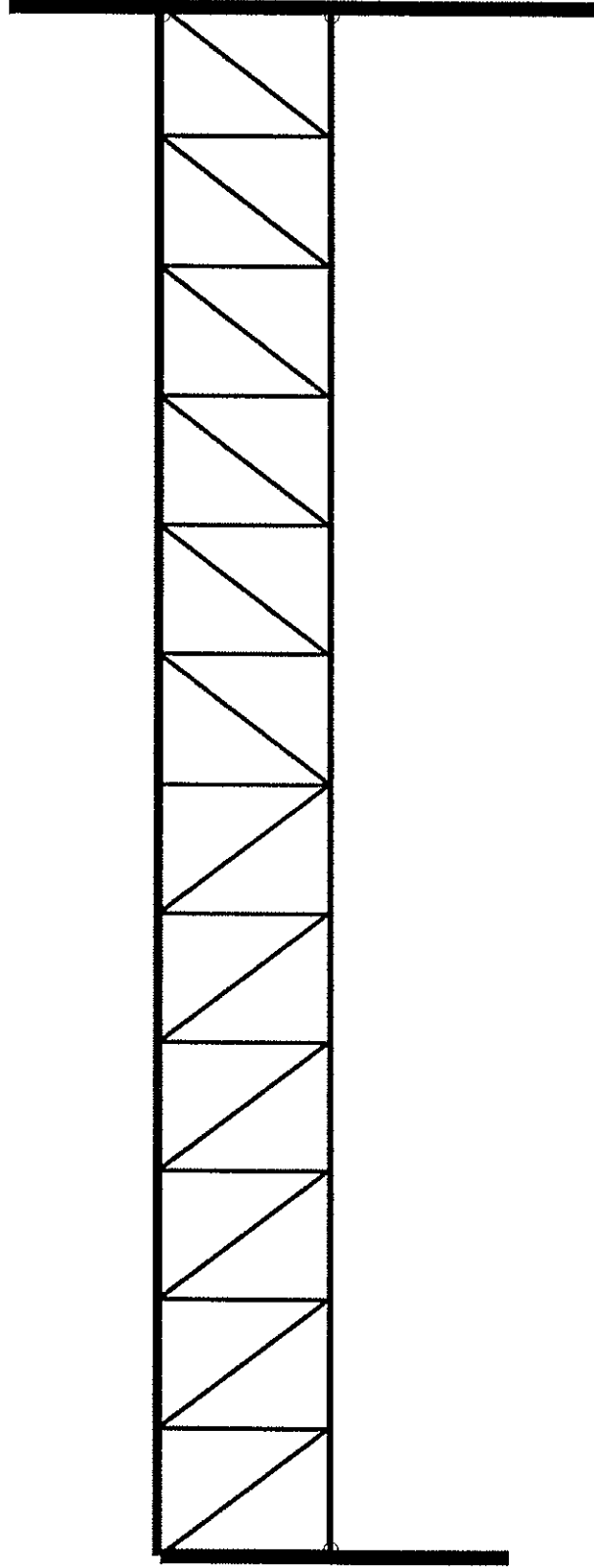
19-Sep-2005 14:08

WhittierTruss.std

1

Rev

Client



Y
Z-X

Load 1

STAAD SPACE TYPICAL WHITTIER TRUSS

START JOB INFORMATION

ENGINEER DATE 09-Sep-05

END JOB INFORMATION

INPUT WIDTH 79

UNIT FEET KIP

JOINT COORDINATES

1 0 0 0; 2 68 0 0; 3 5.66667 0 0; 4 11.3333 0 0; 5 17 0 0; 6 22.6667 0 0;
7 28.3333 0 0; 8 34 0 0; 9 39.6667 0 0; 10 45.3333 0 0; 11 51 0 0;
12 56.6667 0 0; 13 62.3333 0 0; 14 0 7.583 0; 15 5.66667 7.583 0;
16 11.3333 7.583 0; 17 17 7.583 0; 18 22.6667 7.583 0; 19 28.3333 7.583 0;
20 34 7.583 0; 21 39.6667 7.583 0; 22 45.3333 7.583 0; 23 51 7.583 0;
24 56.6667 7.583 0; 25 62.3333 7.583 0; 26 68 7.583 0; 27 0 -14.64 0;
28 68 -12 0; 29 68 14.31 0;

MEMBER INCIDENCES

1 1 3; 2 3 4; 3 4 5; 4 5 6; 5 6 7; 6 7 8; 7 8 9; 8 9 10; 9 10 11; 10 11 12;
11 12 13; 12 13 2; 13 14 15; 14 15 16; 15 16 17; 16 17 18; 17 18 19; 18 19 20;
19 20 21; 20 21 22; 21 22 23; 22 23 24; 23 24 25; 24 25 26; 25 1 14; 26 14 3;
27 3 15; 28 15 4; 29 4 16; 30 16 5; 31 5 17; 32 17 6; 33 6 18; 34 18 7;
35 7 19; 36 19 8; 37 8 20; 38 8 21; 39 21 9; 40 9 22; 41 22 10; 42 10 23;
43 23 11; 44 11 24; 45 24 12; 46 12 25; 47 25 13; 48 13 26; 49 26 2; 50 27 1;
51 28 2; 52 26 29;

DEFINE MATERIAL START

ISOTROPIC STEEL

E 4.176e+006

POISSON 0.3

DENSITY 0.489024

ALPHA 6.5e-006

DAMP 0.03

END DEFINE MATERIAL

CONSTANTS

MATERIAL STEEL MEMB 1 TO 52

MEMBER PROPERTY AMERICAN

13 TO 24 TABLE LD L50354 SP 0.03125

1 TO 12 TABLE SD L40354 SP 0.03125

26 TO 48 TABLE SD L25254 SP 0.03125

25 49 TO 52 TABLE ST W8X28

SUPPORTS

27 28 PINNED

6 9 15 TO 25 FIXED BUT FX FY MX MY MZ

2 29 FIXED BUT FY MX MY MZ

1 FIXED BUT FX FY MX MY MZ

MEMBER TRUSS

26 TO 48

MEMBER RELEASE

12 24 25 END MZ

1 START MZ

LOAD 1 DEAD

JOINT LOAD

15 TO 25 FY -2.8

14 26 FY -1.4

LOAD 2 LIVE

JOINT LOAD

15 TO 25 FY -2.84

14 26 FY -1.3

LOAD 3 SNOW DRIFT

JOINT LOAD

26 FY -3.5

25 FY -5.7

24 FY -3

23 FY -0.3

LOAD 4 SEISMIC

JOINT LOAD

26 FX 1.81

LOAD COMB 10 D+L+S

1 1.0 2 1.0 3 1.0

PERFORM ANALYSIS

LOAD LIST 4 10

PRINT SUPPORT REACTION ALL

PARAMETER

CODE AISC

CHECK CODE ALL

FU 8352 ALL

FYLD 5184 ALL

kX 0 MEMB 1 TO 49

kY 0 MEMB 1 TO 49

lX 1 MEMB 1 TO 26 28 30 to 44 46 48 49
lY 1 MEMB 1 TO 26 28 30 to 44 46 48 49
lZ 1 MEMB 1 TO 26 28 30 to 44 46 48 49
lX 0.85 MEMB 27 29 45 47
lY 0.85 MEMB 27 29 45 47
lZ 0.85 MEMB 27 29 45 47
TRACK 1 MEMB 1 TO 49
FINISH

STAAD SPACE WHITTIER BRACING ALONG COLUMN LINE 1 [NORTH SIDE]

START JOB INFORMATION

JOB NAME Whittier

JOB CLIENT City of Columbus

JOB NO 41616

JOB PART New Bracing on South Side

ENGINEER DATE 13-Sep-05

END JOB INFORMATION

INPUT WIDTH 79

UNIT FEET KIP

JOINT COORDINATES

1 0 0 0; 2 0 14.33 0; 3 20 0 0; 4 20 14.33 0; 5 0 6.21 0; 6 20 6.21 0;
7 10 6.21 0;

MEMBER INCIDENCES

1 1 5; 2 5 2; 3 3 6; 4 6 4; 5 2 4; 6 5 7; 7 7 6; 8 2 7; 11 7 4;

DEFINE MATERIAL START

ISOTROPIC STEEL

E 4.176e+006

POISSON 0.3

DENSITY 0.489024

ALPHA 6.5e-006

DAMP 0.03

END DEFINE MATERIAL

CONSTANTS

BETA 90 MEMB 1 TO 5

MATERIAL STEEL MEMB 1 TO 8 11

MEMBER PROPERTY AMERICAN

8 11 TABLE ST L40404

6 7 TABLE ST L60407

1 TO 4 TABLE ST W8X28

5 TABLE ST C8X11

SUPPORTS

1 3 FIXED

2 4 FIXED BUT FX FY MX MY MZ

5 6 fixed but fy fz mx my mz

MEMBER RELEASE

5 START MZ

5 8 END MZ

6 START MZ

7 END MZ

8 11 START MZ

11 END MZ

LOAD 1 "UNFACTORED" SEISMIC LOAD

JOINT LOAD

2 FX 13

LOAD 2 COLUMN D LOAD

JOINT LOAD

2 4 FY -5.97

LOAD 3 COLUMN L LOAD

JOINT LOAD

2 4 FY -5.2

LOAD COMB 10 0.6D+0.7E

1 1.0 2 0.6

LOAD COMB 11 D+L+0.7E

1 1.0 2 1.0 3 1.0

PDELTA 2 ANALYSIS PRINT LOAD DATA

LOAD LIST 10 11

PARAMETER

CODE AISC

FYLD 5184 MEMB 1 TO 8 11

FU 8352 MEMB 1 TO 8 11

LX 0 MEMB 1 TO 8 11

LY 0 MEMB 1 TO 8 11

LZ 0 MEMB 1 TO 8 11

KX 1 MEMB 1 TO 8 11

KY 1 MEMB 1 TO 8 11

KZ 1 MEMB 1 TO 8 11

CHECK CODE MEMB 1 TO 4 6 TO 8 11

FINISH

STAAD SPACE WHITTIER BRACING ALONG COLUMN LINE 1 [NORTH SIDE]

START JOB INFORMATION

JOB NAME Whittier

JOB CLIENT City of Columbus

JOB NO 41616

JOB PART Bracing on North Side

ENGINEER DATE 13-Sep-05

END JOB INFORMATION

INPUT WIDTH 79

UNIT FEET KIP

JOINT COORDINATES

1 0 0 0; 2 0 14.33 0; 3 20 0 0; 4 20 14.33 0; 5 0 6.21 0; 6 20 6.21 0;

7 10 6.21 0; 8 5 10.27 0; 9 15 10.27 0;

MEMBER INCIDENCES

1 1 5; 2 5 2; 3 3 6; 4 6 4; 5 2 4; 6 5 7; 7 7 6; 8 2 8; 9 7 9; 10 8 7; 11 9 4;

12 5 8; 13 6 9;

DEFINE MATERIAL START

ISOTROPIC STEEL

E 4.176e+006

POISSON 0.3

DENSITY 0.489024

ALPHA 6.5e-006

DAMP 0.03

END DEFINE MATERIAL

CONSTANTS

BETA 90 MEMB 1 TO 5

MATERIAL STEEL MEMB 1 TO 13

MEMBER PROPERTY AMERICAN

8 TO 13 TABLE ST L25255

6 7 TABLE ST L60407

1 TO 4 TABLE ST W8X28

5 TABLE ST C8X11

SUPPORTS

1 3 FIXED

2 4 FIXED BUT FX FY MX MY MZ

MEMBER TRUSS

12 13

MEMBER RELEASE

5 START MZ

5 END MZ

6 START MZ

7 END MZ

8 9 START MZ

10 11 END MZ

LOAD 1 "UNFACTORED" SEISMIC LOAD

JOINT LOAD

2 FX 13

LOAD 2 COLUMN D LOAD

JOINT LOAD

2 4 FY -5.97

LOAD 3 COLUMN L LOAD

JOINT LOAD

2 4 FY -5.2

LOAD COMB 10 0.6D+0.7E

1 1.0 2 0.6

LOAD COMB 11 D+L+0.7E

1 1.0 2 1.0 3 1.0

PDELTA 2 ANALYSIS PRINT LOAD DATA

LOAD LIST 10 11

PARAMETER

CODE AISC

FYLD 5184 MEMB 1 TO 13

FU 8352 MEMB 1 TO 13

LX 0 MEMB 1 TO 13

LY 0 MEMB 1 TO 13

LZ 0 MEMB 1 TO 13

KX 1 MEMB 1 TO 13

KY 1 MEMB 1 TO 13

KZ 1 MEMB 1 TO 13

CHECK CODE MEMB 1 TO 4 6 TO 13

FINISH

STAAD SPACE WHITTIER BRACING ALONG COLUMN LINE 1 [NORTH SIDE]

START JOB INFORMATION

JOB NAME Whittier

JOB CLIENT City of Columbus

JOB NO 41616

JOB PART Existing Bracing on South Side

ENGINEER DATE 13-Sep-05

END JOB INFORMATION

INPUT WIDTH 79

UNIT FEET KIP

JOINT COORDINATES

1 0 0 0; 2 0 22.5 0; 3 20 0 0; 4 20 22.5 0; 5 0 14.92 0; 6 20 14.92 0;

7 10 14.92 0; 8 10 22.5 0;

MEMBER INCIDENCES

1 1 5; 2 5 2; 3 3 6; 4 6 4; 5 2 8; 6 5 7; 7 7 6; 8 2 7; 11 7 4; 12 1 6; 13 8 4;

14 7 8;

DEFINE MATERIAL START

ISOTROPIC STEEL

E 4.176e+006

POISSON 0.3

DENSITY 0.489024

ALPHA 6.5e-006

DAMP 0.03

END DEFINE MATERIAL

CONSTANTS

BETA 90 MEMB 1 TO 5 13

MATERIAL STEEL MEMB 1 TO 8 11 TO 14

MEMBER PROPERTY AMERICAN

8 11 12 14 TABLE ST L25254

1 TO 4 TABLE ST W8X28

5 TO 7 13 TABLE ST C8X11

SUPPORTS

1 3 pinned

2 4 FIXED BUT FX FY MX MY MZ

MEMBER RELEASE

5 START MZ

8 13 END MZ

6 START MZ

7 END MZ

8 11 START MZ

11 END MZ

MEMBER TRUSS

12 14

LOAD 1 "UNFACTORED" SEISMIC LOAD

JOINT LOAD

2 FX 30.5

LOAD 2 COLUMN D LOAD

JOINT LOAD

2 4 FY -17.4

LOAD 3 COLUMN L LOAD

JOINT LOAD

2 4 FY -10.5

LOAD COMB 10 0.6D+0.7E

1 1.0 2 0.6

LOAD COMB 11 D+L+0.7E

1 1.0 2 1.0 3 1.0

PDELTA 2 ANALYSIS PRINT LOAD DATA

LOAD LIST 10 11

PARAMETER

CODE AISC

FYLD 5184 MEMB 1 TO 8 11 TO 14

FU 8352 MEMB 1 TO 8 11 TO 14

LX 0 MEMB 1 TO 8 11 TO 14

LY 0 MEMB 1 TO 8 11 TO 14

LZ 0 MEMB 1 TO 8 11 TO 14

KX 1 MEMB 1 TO 8 11 13 14

KY 1 MEMB 1 TO 8 11 13 14

KZ 1 MEMB 1 TO 8 11 13 14

KX 0.5 MEMB 12

KY 0.5 MEMB 12

KZ 0.5 MEMB 12

CHECK CODE MEMB 1 TO 4 6 TO 8 11 12 14

FINISH

STAAD SPACE WHITTIER BRACING ALONG COLUMN LINE 1 [NORTH SIDE]

START JOB INFORMATION

JOB NAME Whittier

JOB CLIENT City of Columbus

JOB NO 41616

JOB PART Bracing on North Side

ENGINEER DATE 13-Sep-05

END JOB INFORMATION

INPUT WIDTH 79

UNIT FEET KIP

JOINT COORDINATES

1 0 0 0; 2 0 14.33 0; 3 20 0 0; 4 20 14.33 0; 5 0 6.21 0; 6 20 6.21 0;

7 10 6.21 0; 8 5 10.27 0; 9 15 10.27 0;

MEMBER INCIDENCES

1 1 5; 2 5 2; 3 3 6; 4 6 4; 5 2 4; 6 5 7; 7 7 6; 8 2 8; 9 7 9; 10 8 7; 11 9 4;

12 5 8; 13 6 9;

DEFINE MATERIAL START

ISOTROPIC STEEL

E 4.176e+006

POISSON 0.3

DENSITY 0.489024

ALPHA 6.5e-006

DAMP 0.03

END DEFINE MATERIAL

CONSTANTS

BETA 90 MEMB 1 TO 5

MATERIAL STEEL MEMB 1 TO 13

MEMBER PROPERTY AMERICAN

8 TO 13 TABLE ST L25255

6 7 TABLE ST L60407

1 TO 4 TABLE ST W8X28

5 TABLE ST C8X11

SUPPORTS

1 3 FIXED

2 4 FIXED BUT FX FY MX MY MZ

7 FIXED BUT FY FZ MX MY mz

MEMBER TRUSS

12 13

MEMBER RELEASE

5 START MZ

5 END MZ

6 START MZ

7 END MZ

8 9 START MZ

10 11 END MZ

LOAD 1 "UNFACTORED" SEISMIC LOAD

JOINT LOAD

2 FX 13

LOAD 2 COLUMN D LOAD

JOINT LOAD

2 4 FY -5.97

LOAD 3 COLUMN L LOAD

JOINT LOAD

2 4 FY -5.2

LOAD COMB 10 0.6D+0.7E

1 1.0 2 0.6

LOAD COMB 11 D+L+0.7E

1 1.0 2 1.0 3 1.0

PDELTA 2 ANALYSIS PRINT LOAD DATA

LOAD LIST 10 11

PARAMETER

CODE AISC

FYLD 5184 MEMB 1 TO 13

FU 8352 MEMB 1 TO 13

LX 0 MEMB 1 TO 13

LY 0 MEMB 1 TO 13

LZ 0 MEMB 1 TO 13

KX 1 MEMB 1 TO 13

KY 1 MEMB 1 TO 13

KZ 1 MEMB 1 TO 13

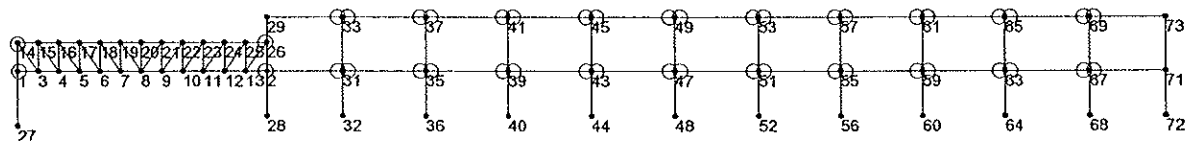
CHECK CODE MEMB 1 TO 4 6 TO 13

FINISH



Software licensed to Burges

Job No	Sheet No 1	Rev
Part		
Ref		
By	Date09-Sep-05	Chd
Client	File WhittierNorthSouthMome	Date/Time 23-Sep-2005 09:47



Y
Z-X

Load 10

STAAD SPACE TYPICAL WHITTIER TRUSS

START JOB INFORMATION

ENGINEER DATE 09-Sep-05

END JOB INFORMATION

INPUT WIDTH 79

UNIT FEET KIP

JOINT COORDINATES

1 0 0 0; 2 68 0 0; 3 5.66667 0 0; 4 11.3333 0 0; 5 17 0 0; 6 22.6667 0 0;
 7 28.3333 0 0; 8 34 0 0; 9 39.6667 0 0; 10 45.3333 0 0; 11 51 0 0;
 12 56.6667 0 0; 13 62.3333 0 0; 14 0 7.583 0; 15 5.66667 7.583 0;
 16 11.3333 7.583 0; 17 17 7.583 0; 18 22.6667 7.583 0; 19 28.3333 7.583 0;
 20 34 7.583 0; 21 39.6667 7.583 0; 22 45.3333 7.583 0; 23 51 7.583 0;
 24 56.6667 7.583 0; 25 62.3333 7.583 0; 26 68 7.583 0; 27 0 -14.64 0;
 28 68 -12 0; 29 68 14.31 0; 31 88 0 0; 32 88 -12 0; 33 88 14.31 0;
 35 110.17 0 0; 36 110.17 -12 0; 37 110.17 14.31 0; 39 132.33 0 0;
 40 132.33 -12 0; 41 132.33 14.31 0; 43 154.5 0 0; 44 154.5 -12 0;
 45 154.5 14.31 0; 47 176.67 0 0; 48 176.67 -12 0; 49 176.67 14.31 0;
 51 198.85 0 0; 52 198.85 -12 0; 53 198.85 14.31 0; 55 221 0 0; 56 221 -12 0;
 57 221 14.31 0; 59 243.17 0 0; 60 243.17 -12 0; 61 243.17 14.31 0;
 63 265.33 0 0; 64 265.33 -12 0; 65 265.33 14.31 0; 67 287.5 0 0;
 68 287.5 -12 0; 69 287.5 14.31 0; 71 307.5 0 0; 72 307.5 -12 0;
 73 307.5 14.31 0;

MEMBER INCIDENCES

1 1 3; 2 3 4; 3 4 5; 4 5 6; 5 6 7; 6 7 8; 7 8 9; 8 9 10; 9 10 11; 10 11 12;
 11 12 13; 12 13 2; 13 14 15; 14 15 16; 15 16 17; 16 17 18; 17 18 19; 18 19 20;
 19 20 21; 20 21 22; 21 22 23; 22 23 24; 23 24 25; 24 25 26; 25 1 14; 26 14 3;
 27 3 15; 28 15 4; 29 4 16; 30 16 5; 31 5 17; 32 17 6; 33 6 18; 34 18 7;
 35 7 19; 36 19 8; 37 8 20; 38 8 21; 39 21 9; 40 9 22; 41 22 10; 42 10 23;
 43 23 11; 44 11 24; 45 24 12; 46 12 25; 47 25 13; 48 13 26; 49 26 2; 50 27 1;
 51 28 2; 52 26 29; 53 33 31; 54 32 31; 56 37 35; 57 36 35; 59 41 39; 60 40 39;
 62 45 43; 63 44 43; 65 49 47; 66 48 47; 68 53 51; 69 52 51; 71 57 55; 72 56 55;
 74 61 59; 75 60 59; 77 65 63; 78 64 63; 80 69 67; 81 68 67; 83 73 71; 84 72 71;
 86 29 33; 87 33 37; 88 37 41; 89 41 45; 90 45 49; 91 49 53; 92 53 57; 93 57 61;
 94 61 65; 95 65 69; 96 69 73; 97 2 31; 98 31 35; 99 35 39; 100 39 43;
 101 43 47; 102 47 51; 103 51 55; 104 55 59; 105 59 63; 106 63 67; 107 67 71;

DEFINE MATERIAL START

ISOTROPIC STEEL

E 4.176e+006

POISSON 0.3

DENSITY 0.489024

ALPHA 6.5e-006

DAMP 0.03

END DEFINE MATERIAL

CONSTANTS

MATERIAL STEEL MEMB 1 TO 54 56 57 59 60 62 63 65 66 68 69 71 72 74 75 77 78 -
 80 81 83 84 86 TO 107

MEMBER PROPERTY AMERICAN

13 TO 24 TABLE LD L50354 SP 0.03125

1 TO 12 TABLE SD L40354 SP 0.03125

26 TO 48 TABLE SD L25254 SP 0.03125

25 49 TO 54 56 57 59 60 62 63 65 66 68 69 71 72 74 75 77 78 80 81 83 -

84 TABLE ST W8X28

86 TO 96 TABLE ST W12X30

97 TO 107 TABLE ST W18X55

SUPPORTS

27 28 32 36 40 44 48 52 56 60 64 68 72 PINNED

6 9 15 TO 25 FIXED BUT FX FY MX MY MZ

2 29 31 33 35 37 39 41 43 45 47 49 51 53 55 57 59 61 63 65 67 69 71 -

73 FIXED BUT FX FY MX MY MZ

1 FIXED BUT FX FY MX MY MZ

MEMBER TRUSS

26 TO 48

MEMBER RELEASE

12 24 25 END MZ

1 START MZ

87 TO 97 START MZ

86 TO 95 END MZ

98 TO 107 START MZ

97 TO 107 END MZ

LOAD 1 DEAD

JOINT LOAD

15 TO 25 FY -2.8

14 26 FY -1.4

** upper roof

29 73 FY -5.8

33 69 FY -6.6

```

** 2nd floor
2 71 FY -14.7
31 67 FY -28.6
35 39 43 47 51 55 59 63 FY -29.8
LOAD 2 LIVE
JOINT LOAD
15 TO 25 FY -2.84
14 26 FY -1.3
** upper roof
29 73 FY -4.2
33 69 FY -11.3
37 41 45 49 53 57 61 65 FY -12
** 2nd floor
2 71 FY -7.9
31 67 FY -13
35 39 43 47 51 55 59 63 FY -13.4
LOAD 3 SNOW DRIFT
JOINT LOAD
26 FY -3.5
25 FY -5.7
24 FY -3
23 FY -0.3
LOAD 4 SEISMIC (ALREADY UNFACTORED)
JOINT LOAD
73 FX 9.8
26 FX 2.4
71 FX 10.9
LOAD 5 SEISMIC (ALREADY UNFACTORED)
JOINT LOAD
73 FX -9.8
26 FX -2.4
71 FX -10.9
LOAD COMB 10 D+S+0.7En
1 1.0 2 1.0 3 1.0 4 1.0
LOAD COMB 11 D+S+0.7Es
1 1.0 2 1.0 3 1.0 5 1.0
PERFORM ANALYSIS
PDELTA 2 ANALYSIS
LOAD LIST 10 11
PRINT SUPPORT REACTION ALL
PARAMETER
CODE AISC
FU 8352 ALL
FYLD 5184 ALL
CHECK CODE MEMB 49 TO 54 56 57 59 60 62 63 65 66 68 69 71 72 74 75 77 78 80 -
81 83 84 86 TO 107
FINISH

```